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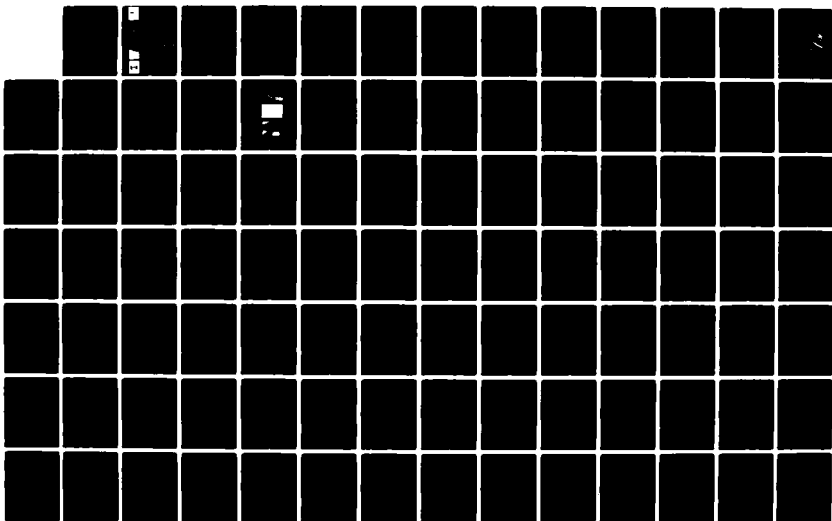
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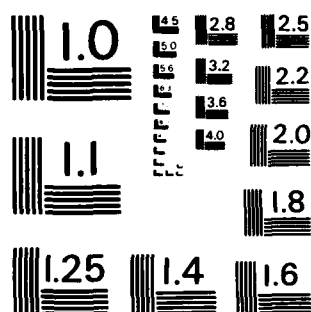
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COMPUTER MODELING OF HYDRODYNAMICS
AND SOLUTE TRANSPORT
IN CANALS AND MARINAS

LITERATURE REVIEW AND GUIDELINES
FOR FUTURE DEVELOPMENT

by

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Final Report

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PREFACE

The work described in this report was conducted by Dr. Raymond Walton, Camp Dresser & McKee, for the U. S. Army Engineer Waterways Experiment Station (WES) under Contract No. DACW39-82-M-2659. The study was sponsored by the Office, Chief of Engineers, U. S. Army, under the Environmental Impact Research Program (EIRP).

The purpose of the work was to review the literature and provide guidelines for future developments to model water quality in marinas and canals. The intended audience is Corps of Engineer personnel with a need for information on numerical modeling.

Many people contributed to the work in this report through discussions. The author would like to thank Dr. Raymond Chapman of Oceanweather, Inc.; Dr. B. A. Christensen and Mr. Don Hayes of the Hydraulic Laboratory, University of Florida; Mr. Robert M. Snyder of Snyder Oceanography, Jupiter, FL; Dr. Fred Morris of the South Florida Water Management District; Dr. J. van de Kreeke of RSMAS, University of Miami; Mr. Tom Cavinder of the Environmental Protection Agency (EPA), Athens, GA; Dr. Billy Edge of Cubit Engineering, Clemson, SC; Dr. Larry Slotta of Oregon State University; Mr. Rick Swartz and Mr. R. J. Callaway of EPA, Corvallis, OR; and Dr. Ron Nece of the University of Washington, WA; for their many helpful thoughts on the present state of the art and the future direction of canal and marina modeling.

Mr. Ross Hall was the EIRP Principal Investigator. General supervision was provided by Mr. Donald L. Robey, Chief, Ecosystem Research and Simulation Division, WES, and Dr. John Harrison, Chief, Environmental Laboratory (EL), WES. Program Manager of EIRP was Dr. Roger Saucier, EL. Mr. John Bushman was Technical Monitor for the Office, Chief of Engineers.

Commander and Director of WES during the preparation of this report was COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

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Acres	4046.873	Square meters
Cubic feet per day per foot	0.0929	Cubic meters per day per meter
Feet	0.3048	Meters
Feet per second	0.0348	Meters per second
Horsepower (550 foot-pounds per second)	745.6999	Watts
Miles per hour (U. S. statute)	1.609347	Kilometers per hour
Square feet	0.09290304	Square meters
Square feet per second	0.09290304	Square meters per second

COMPUTER MODELING OF HYDRODYNAMICS AND SOLUTE TRANSPORT IN CANALS AND MARINAS

Literature Review and Guidelines for Future Development

I. INTRODUCTION

I.1 Background

Over the past few decades, there has been a tremendous increase in the demand for residential canals and marinas along the coasts of the United States. This demand has led to the design and construction of numerous such systems, often without the benefit of a sound scientific background.

As an example, from the late 1940's to the early 1960's, many large-scale residential waterfront developments were constructed in Florida. The great majority of these developments were branching networks of canals (Figure I.1). They were built by dredging and filling coastal wetlands. By 1974, two developments of over 100,000 acres* each, and 14 over 10,000 acres, were either occupied or under construction. The total land area under development in Florida alone, in 1974, exceeded 810,000 acres (Carter, 1974).

Unfortunately, it is now evident that many of these systems are not the beautiful water bodies anticipated, but rather they are beset with all kinds of problems. Many of these systems flush poorly, have poor water quality, are silting up, or have weak circulation. The same people who demanded that such systems be built are now demanding that scientists and engineers study them in more detail to learn how they work and what constitutes a good design.

* A table of factors for converting U.S. customary units of measurement to metric (SI) is presented on page vi.



Figure I.1. Example of Extensive Residential
Canal Network in Florida

Up to this time, the "science" of canal and marina design in the coastal zone has grown largely by learning from past mistakes. Analytic and numerical models used were often developed for other purposes, and were not always adequate to describe the physics. The contractor's prime motivation was to maximize the number of waterfront dwellings or boat slips.

The problem is further compounded by the permitting process, in which local, State, and Federal agencies must evaluate canal and marina designs and the validity of analytic tools used. Given a lack of understanding of the physics of these systems, and the multitude of "simulation" tools that can and have been used, it is no wonder that the permitting agencies are somewhat confused, and in some cases resort to a position of "no more" as a way out. This is a safe approach because it is easy to suggest that we will negatively impact the environment, as we have in the past.

The design engineer's view, however, is that they can now build better systems that are compatible with the environment, and indeed in some cases may improve on it.

Consequently, within the past few years, much more emphasis has been placed on learning what makes canal and marina systems work. Although the surface has only just been scratched, today there is a better understanding of the physics involved, and thus a better understanding of how to develop the analytical tools needed for canal and marina design.

I.2 Purpose and Scope

The Corps of Engineers, as a permitting agency, must be in an informed position to evaluate proposed canal and marina designs. An understanding of the physical processes at work, and of the ability of existing analytical tools to accurately simulate these processes, is essential. Such judgments must be made in the light of local, State, and Federal regulations. The discussion of present conditions is covered in Chapters II-IV.

One of the purposes of this report is to discuss the present understanding of how these systems work, and then review existing numerical and analytical techniques for simulating hydrodynamic and solute transport processes. To do this, past studies of canal and marina designs and simulations will be reviewed, and such evaluation criteria as flushing rates and residence time will be examined.

As a research and modeling group, the U.S. Army Engineer Waterways Experiment Station (WES) is in a position to extend our ability to understand and model these systems. In this report, future needs in canal and marina design and simulation will be addressed. This, with an approach to further studies, will be the subject of Chapter V.

I.3 Distinction Between Canals and Marinas

In this report, a distinction will be made between the definitions of canals and marinas, although it is realized that such a difference may not always be significant. This is done based on geometric and usage considerations.

Christensen and Snyder (1978) have classified the above types of water bodies into seven major groups (Figure I.2):

1. Simple dead-end canal,
2. Flow-through canal,
3. Comb-structure canal system,
4. Higher order finger canal,
5. Canal with lagoon basin,
6. Lagoon with two tidal entrances, and
7. Borrow lake.

A canal is considered to be a long, narrow waterway, which is primarily used for residence with a docking facility. Such canal systems (numbers 1-4

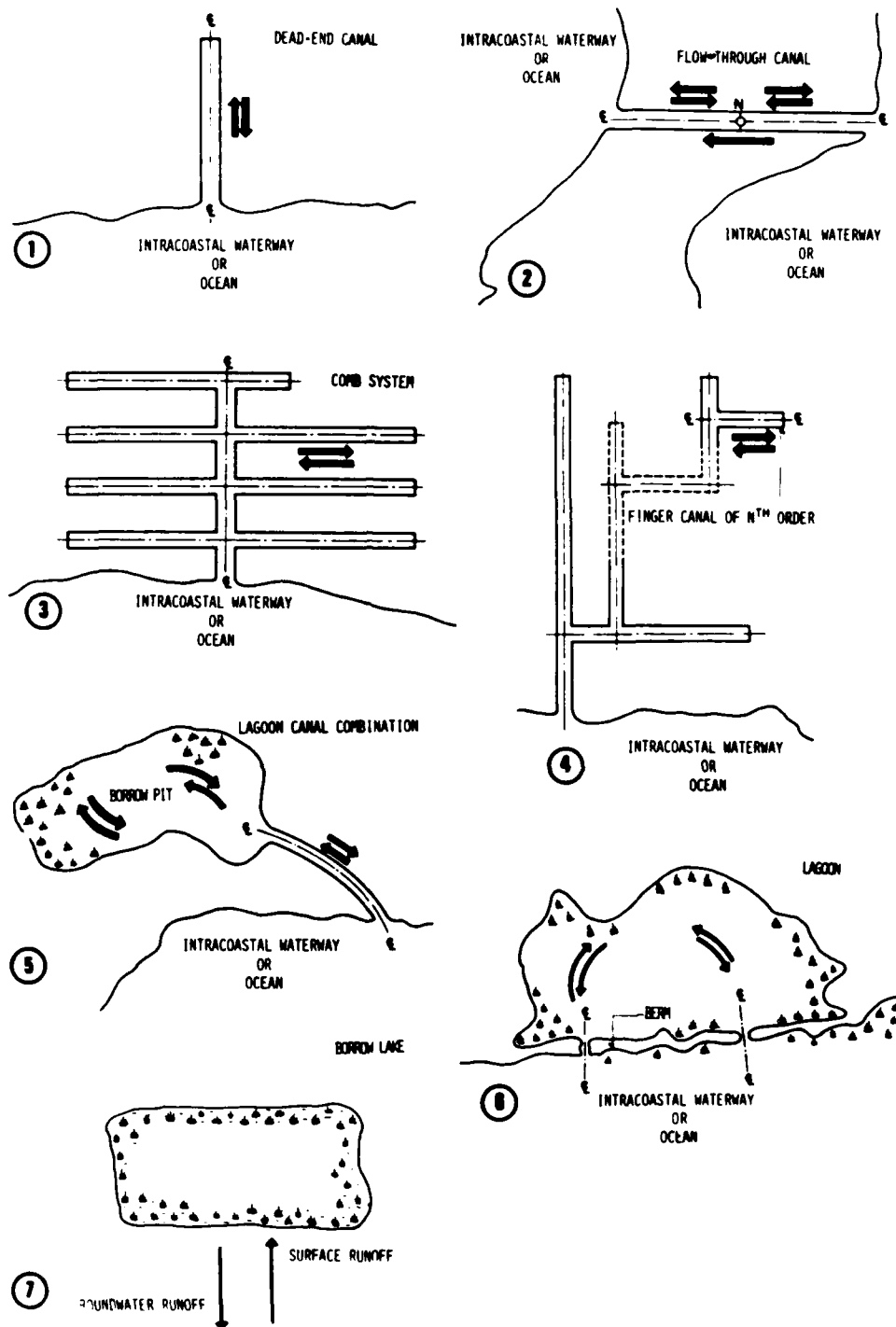


Figure I.2. Classification of Water Bodies

above) are frequently found along the lower Atlantic and Gulf Coast seaboards where the tidal ranges are small. Waterfront canals will be further subdivided into three types (Barada and Partington, 1972; Lindall and Trent, 1975):

1. Bay-fill or finger fill - those constructed below mean low tide by dredging and filling shallow bay bottoms (Figure I.3a),
2. Intertidal - those constructed by dredge-and-fill between mean low and mean high waters. In many cases, these canals are located in mangrove or salt marsh ecosystems, in bays, estuaries, lakes, or other wetlands (Figure I.3b), and
3. Inland or upland - those developed by excavating, which are above mean high water and connected to natural channels, lakes, rivers, or other natural or artificial waterways (Figure I.3c).

Marinas, on the other hand, are primarily used for boat docking. The width is often on the same order of magnitude as the length (Figure I.5). Classifications 5 and 6 in Figure I.2 show typical shapes.



a. Bay - Fill Development



b. Intertidal Development



c. Inland Development

Figure I.3. Waterfront Canals

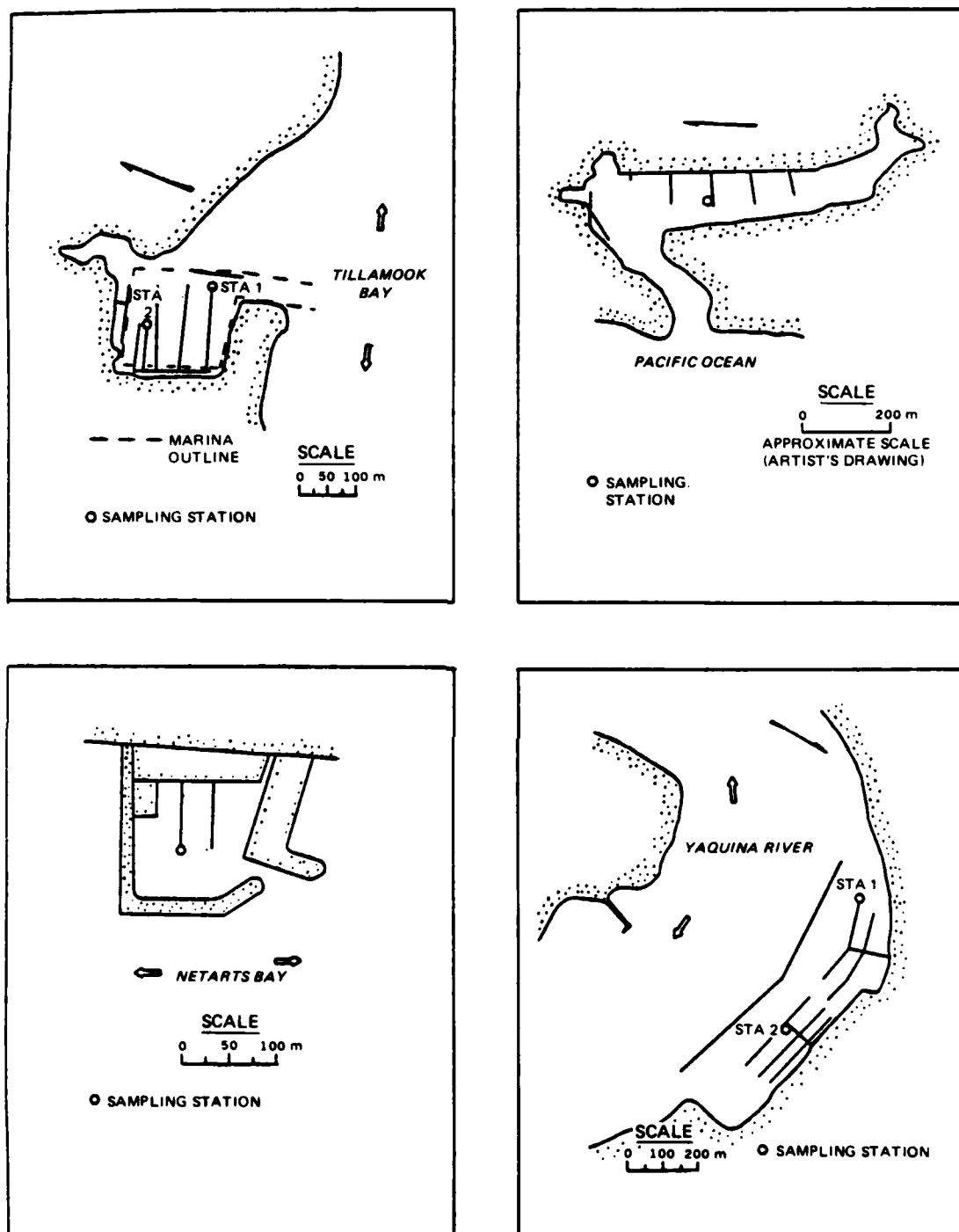


Figure I.4. Schematic Examples of Pacific Northwest Marina Geometries

II. PROCESSES

II.1 Canals

The flows in residential canal systems are complex, time-varying interactions of forcing due to tide, wind, density-induced currents, and secondary currents due to topographic features such as bends and sills. An understanding of these processes, and their relative effects, is crucial in choosing both design approaches and evaluation criteria.

Several different institutions have made fairly comprehensive measurements in canal systems, although such studies tend to concentrate on one or more aspects of canal processes and omit others. Considering hydrodynamics and transport mechanisms, a number of measurements were performed by the Hydraulic Laboratory of the University of Florida during a three-year Sea Grant and other hydrographic studies (University of Florida, 1976; Morris and Christensen, 1976; Morris et al., 1977, 1978a; Walton et al., 1975a, 1975b). Typical canal parameters are listed in Table II.1.

Using tidal range as a measure of available energy to these systems, residential canals along the Florida and Gulf Coasts are classified as low energy canals. Tidal ranges (Figure II.1) are small, as are induced tidal velocities, even at the mouths of many systems. Maximum velocities on the order of 0.5 ft/sec are not uncommon, and these magnitudes decrease towards the dead ends of these systems. Observed maximum water surface slopes were measured to be on the order of 10^{-5} .

Because induced tidal velocities are relatively small, other types of forcing can often play a significant role in canal hydrodynamics. In many of the systems observed (Walton et al., 1975b, Morris et al., 1978a), wind

Table II.1 - Typical Measured Canal Parameters

PARAMETER	RANGE
Length, L	1,500 - 10,000 ft
Bottom width, b	50 - 100 ft
Mean tidal depth, d_0	5 - 12 ft
Inverse side slope, s	0 - 5
Tidal range, r: Atlantic	2 - 5 ft
Gulf of Mexico	2 - 3 ft
Tidal period, T	12.42 hr
Nikuradse's equivalent sand roughness, k	1 - 20 ft
Maximum (in canal) longitudinal dispersion coefficient, E	0.5 - 5.0 ft ² /sec
Dimensionless dispersion coefficient, K	2 - 20
Maximum water surface slope, S	10 ⁻⁵ - 10 ⁻⁶

Table II.2 - Typical Measured Marina Parameters

PARAMETER	RANGE
Length, L	1000 - 3000 ft
Width, b	400 - 1000 ft
Mean lower low water depth	5 - 15 ft
Tidal range, r	3 - 13 ft
Tidal period, T	12.4 hr

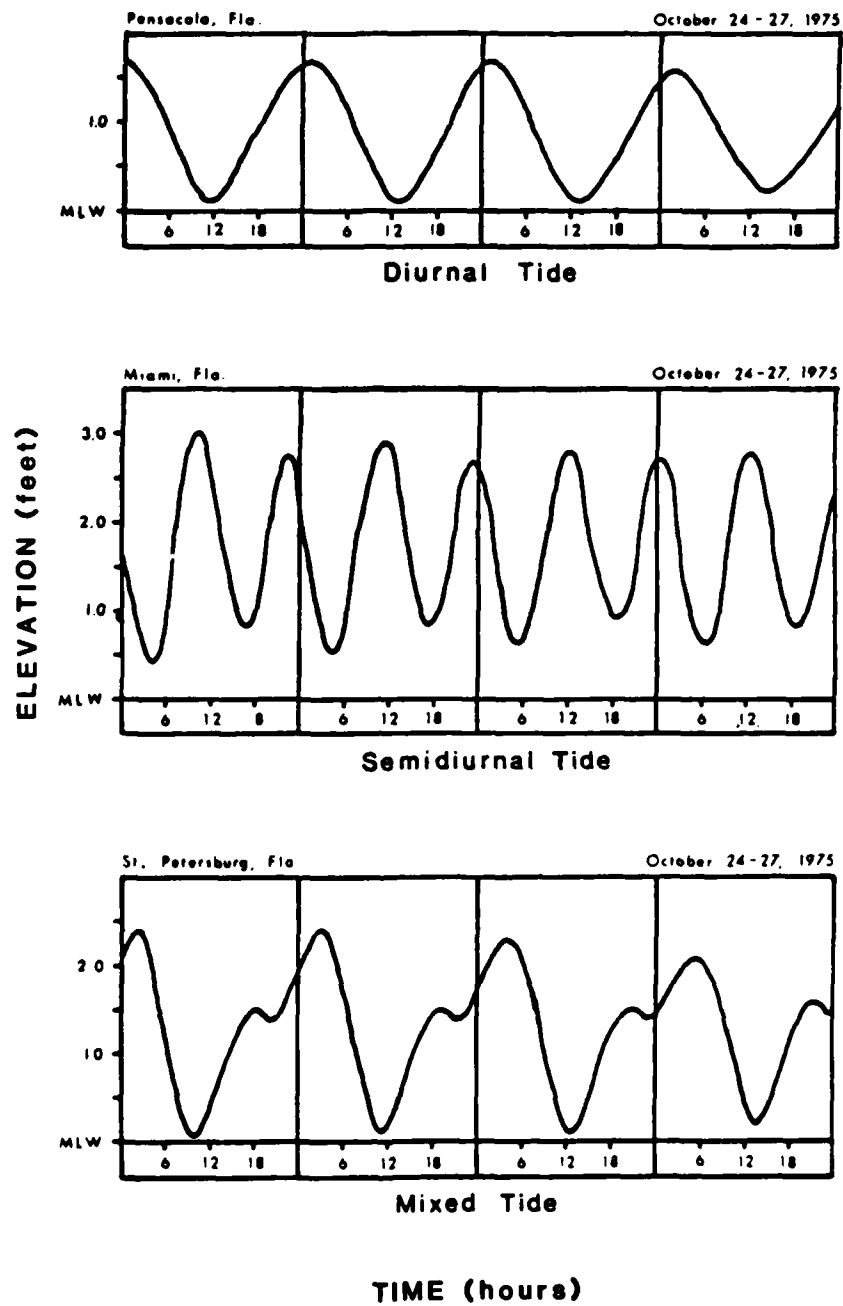


Figure II.1. Typical Tide Curves
in Floridian Canals

was perhaps the dominant short-term driver. When a wind blows along the axis of a dead-end canal system for some period of time, continuity of mass may produce a counterflow in the lower layers (Figure II.2). In a flow-through system, the wind will tend to superimpose on the tidal flow, in the one-dimensional sense of longitudinal transport.

This observation of wind-induced counterflows may be seen in other measurements as well. Dye introduced into a canal system on a windy day may not exhibit the Gaussian distribution expected of one-dimensional, mean flow physics (Figure II.3). In this case, the wind was blowing down the canal producing an observable flow reversal in the lower layers (Figure II.2). This carried some of the dye upstream where it was upwelled or diffused into the surface water to elongate the "tail" of the distribution. This same physical explanation can be given to explain the data of dye studies performed by the Environmental Protection Agency (EPA 1975).

A longer term mechanism, observed particularly in south Florida canals during the hot, wet summer months, is density-induced circulation. McKeehan (1975), van de Kreeke et al. (1977), and Monopolis (1978) have noticed that during these periods of relatively light winds and relatively large freshwater inflows, density stratification can occur (Figure II.4), which induces circulation. Van de Kreeke* believes that these density-induced circulations are important in long-term circulation and water quality transport, particularly during the wet summer months.

Even superimposed, these various types of forcing produce real time velocities whose magnitudes are generally small. Examples of three-layer flows in these shallow systems are not uncommon (Figure II.4). These small velocities in turn lead to relatively low rates of mixing as seen from observed longitudinal dispersion coefficients. Dye studies in a number of canals (Morris et al., 1978a; Walton et al., 1975a, 1975b; EPA, 1973, 1975) have measured coefficients on the order of $5 \text{ ft}^2/\text{sec}$ and less in these systems.

* Personal Communication, J. van de Kreeke, Rosenstiel School of Marine and Atmospheric Science, University of Miami, Coral Gables, FL, 1982.

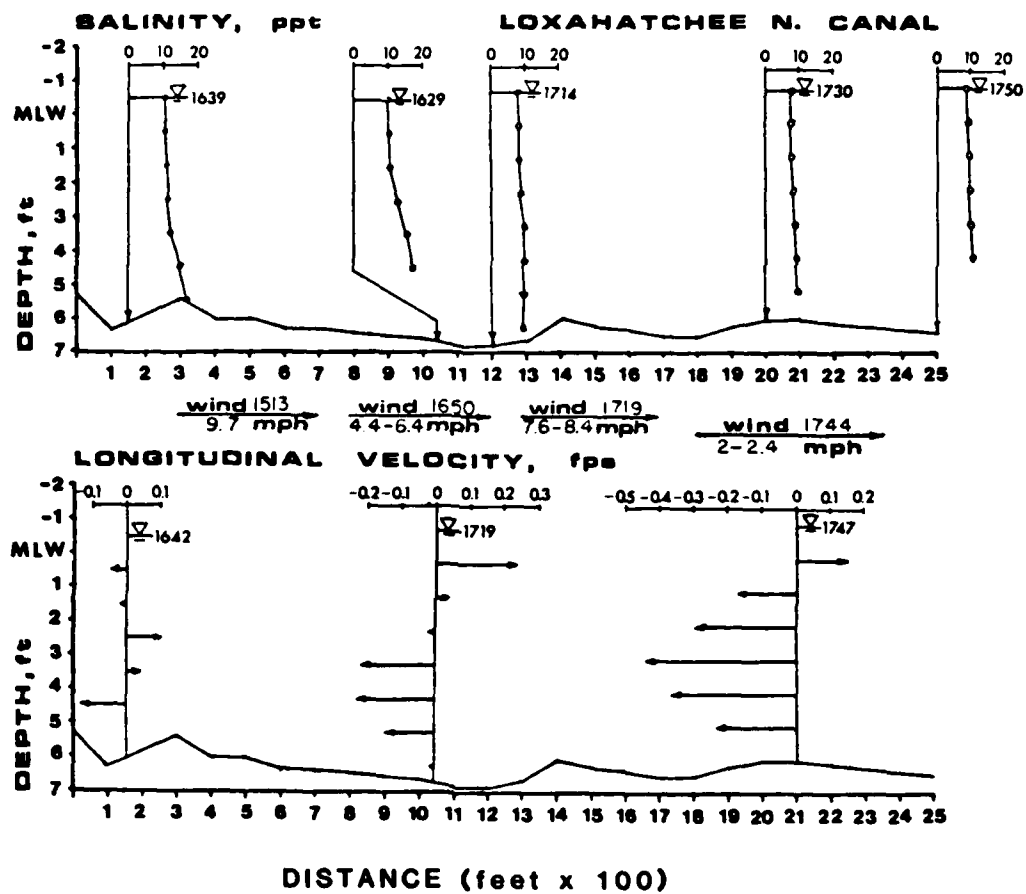


Figure II.2. Salinity and Velocity Measurements Show Flow Reversal in Loxahatchee Canal, Florida

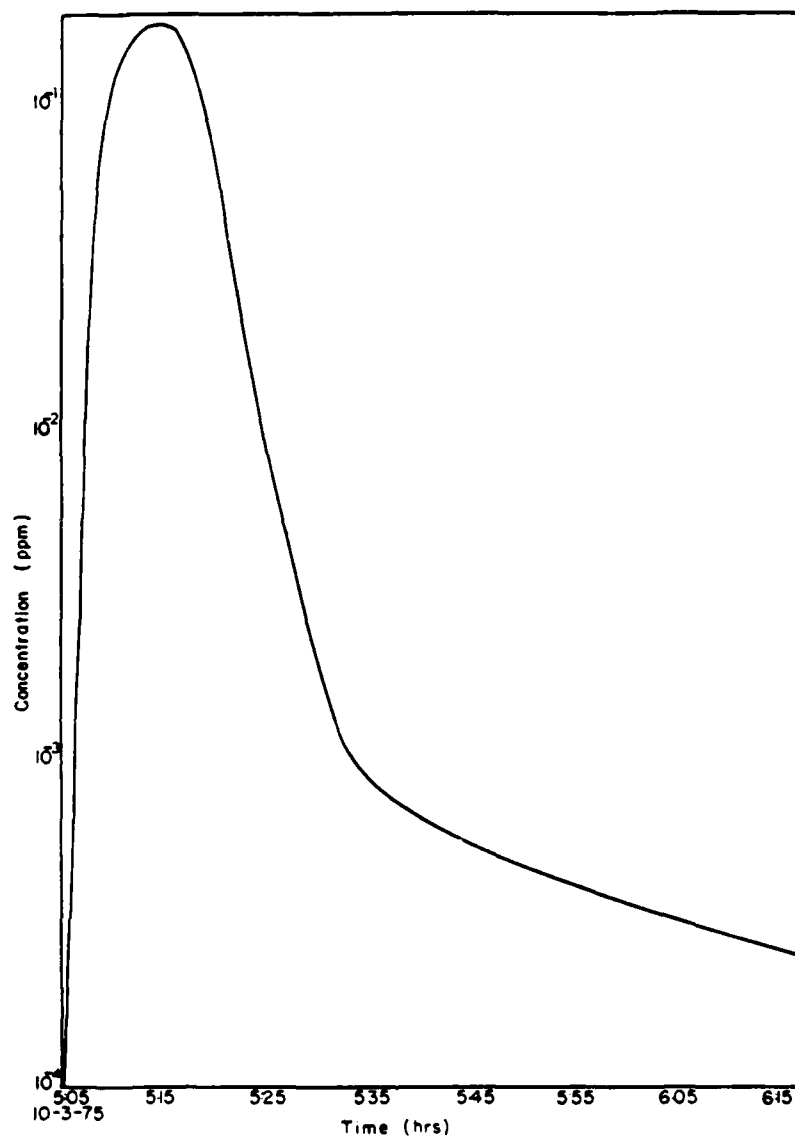


Figure II.3. Non-Gaussian Distribution of Dye Concentration in Floridian Canal

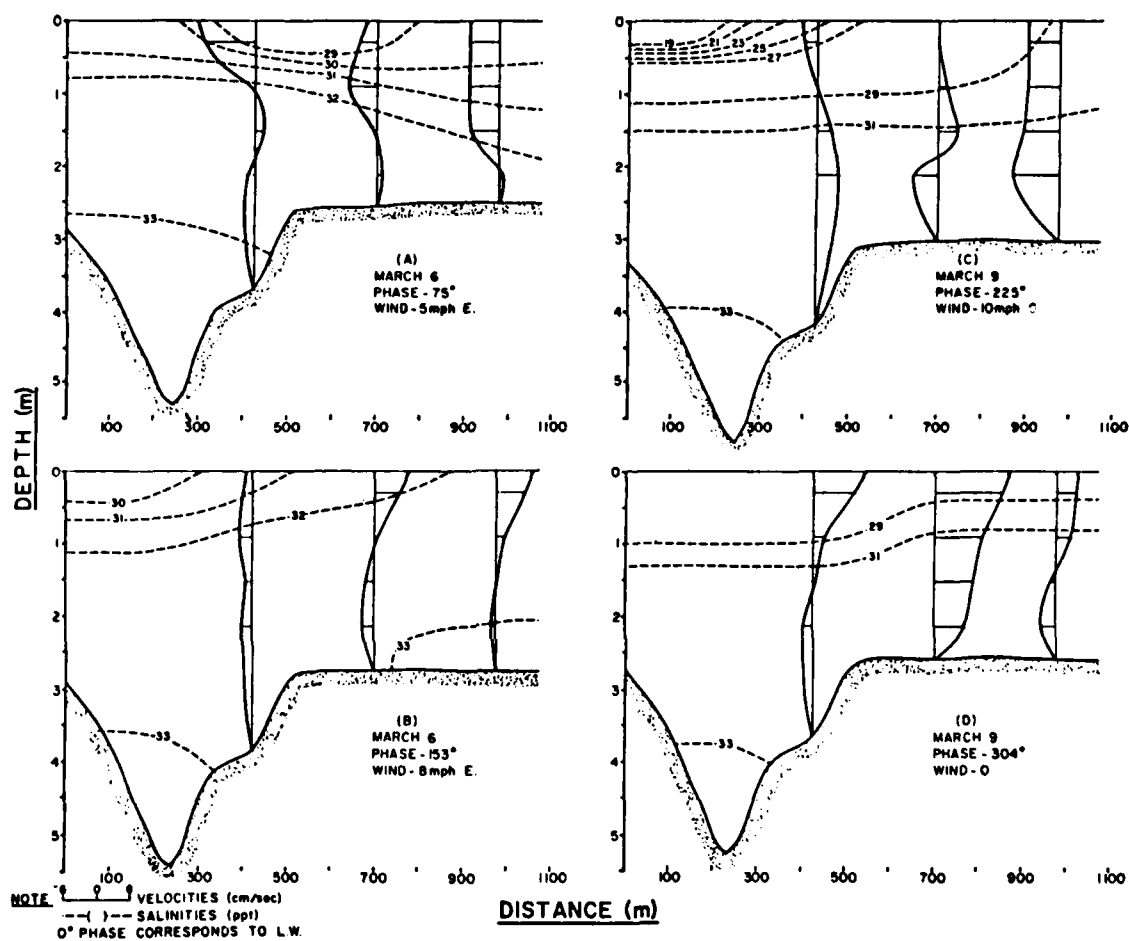


Figure II.4. Salinity and Velocity Distributions in Canal 13-3, Florida (from Monopolis, 1978)

This relatively weak mixing due to variations in the cross-sectioned velocity distribution means that understanding the induced mixing effects of topographic features such as bends, boat docks, branches, and undulating beds becomes important. A number of laboratory studies at the University of Florida (O'Hargan, 1975; Parr and Christensen, 1977; Swenty, 1977) have shown how longitudinal dispersion may be increased by the presence of bends and boat docks. This effect has also been noted in streams (Fischer, 1967, 1969).

It is interesting to note on this subject that, even though these systems are classified as low energy tidal canals, the available energy used in friction and other losses is small, as can be calculated using classical hydraulic formulae. Defining the potential energy, E_p , of the tides in a canal system:

$$E_p = A_s \gamma a^2 \quad (II.1)$$

where A_s = area of water surface (L^2),
 γ = weight density of water (ML/T^2), and
 a = tidal amplitude (L).

Snyder* calculated the available energy to a canal system he was studying to be 1.4 hp. He further calculated that losses due to form drag, bends, constrictions/expansions, pipes/culverts, and friction were only 0.5 percent of the available energy.

In a series of experiments, Morris et al. (1978a) deployed three dual-axis electromagnetic current meters in a cross section of a canal just below a sharp bend. Periodically, vertical profiles were taken at each station and pieced together to produce a cross-sectional view of the velocity field (Figure II.5). From this figure, we can clearly see the induced secondary current, which, if persistent, would overturn the water column, bringing oxygen-depleted waters to the surface.

* Memorandum to Florida Department of Environmental Regulation from Snyder Oceanography Services, Jupiter, FL, 17 June 1978.

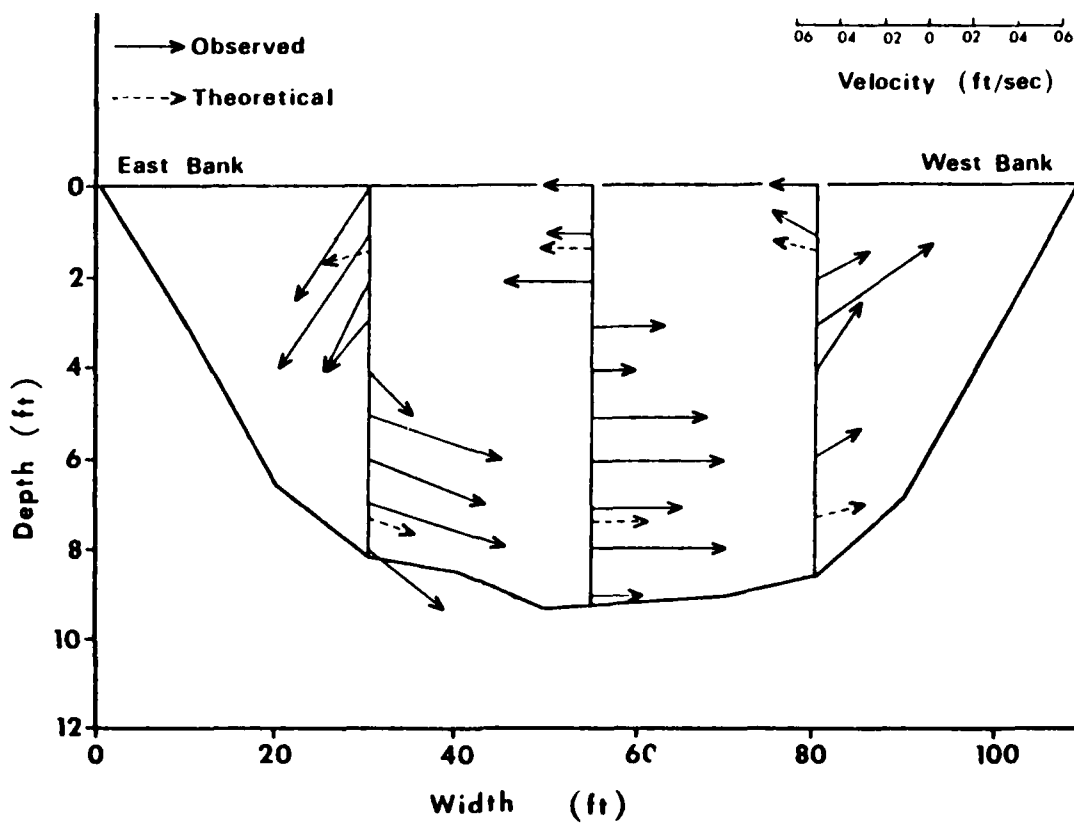


Figure II.5. Secondary Current in 57 Acres Canal System, Florida

A knowledge of canal hydrodynamics is important not only for itself, but also in understanding water quality and sedimentation processes. The literature on the environmental effect of canals and dredged holes contains data on salinity, temperature, and dissolved oxygen which are useful for the design engineer. Some of the literature was first reviewed for a study of the environmental impact of borrow pits in Maryland estuarine waters (Polis, 1974). Polis summarized work in Texas and on the Atlantic Coast from Florida to New Jersey. Since then, much additional work has been performed on studying water quality in tidal canal systems. The EPA (1973, 1975) did perhaps the largest study in canals from Florida to North Carolina. Paulson et al. (1974, 1975, 1977a, 1977b) measured water quality parameters in canals in Mississippi, and Bailey (1977) did some recent work in Florida.

Dissolved oxygen (DO) is often considered to be a standard of canal health, and values anywhere between zero and saturation have been measured. Zero values are often associated with the dead ends of canal systems and deep holes (Lindall et al., 1975 - Figure II.6). Low DO values have been held responsible for fish kills in Floridian canals where it is thought that the fish become trapped and cannot find their way to oxygen rich waters.

The most comprehensive set of DO measurements in canals attempted to relate DO values to the depth of the canal system to determine maximum desired depths in these systems (EPA, 1973, 1975). Results of this analysis are shown in Figures II.7 and II.8 for North Carolinian and Floridian canals, but as can be seen in Figures II.9 and II.10, for two different canal systems in Florida, this result is not universal. Such an analysis provides a reasonable basis for a design regulation. However, more work needs to be done to understand why some systems differ from the general observation, and under what conditions.

In an interesting study of water quality, Messenger and Reynolds (1979) obtained five months of data in a residential canal network on the Texas coast (Figure II.11). They attempted to relate measured values to the ages

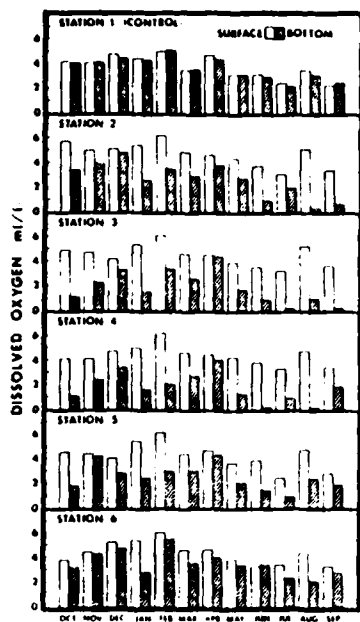


Figure II.6. Monthly DO Readings (from Lindall et al., 1975)

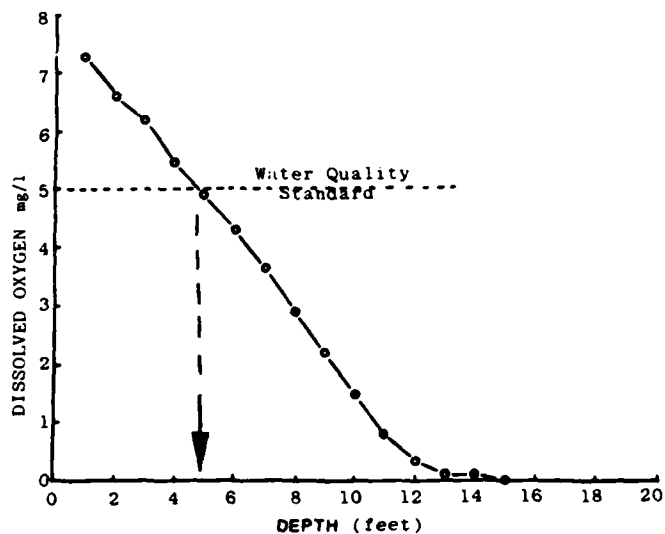


Figure II.7. DO Values in North Carolina Canals Varying With Depth (from EPA, 1975)

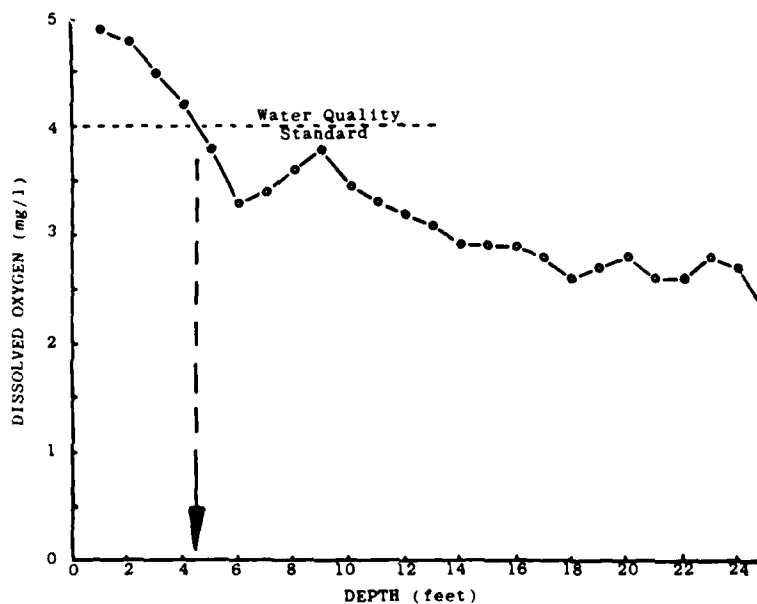


Figure II.8. DO Values in Florida Canals Varying With Depth (from EPA, 1975)

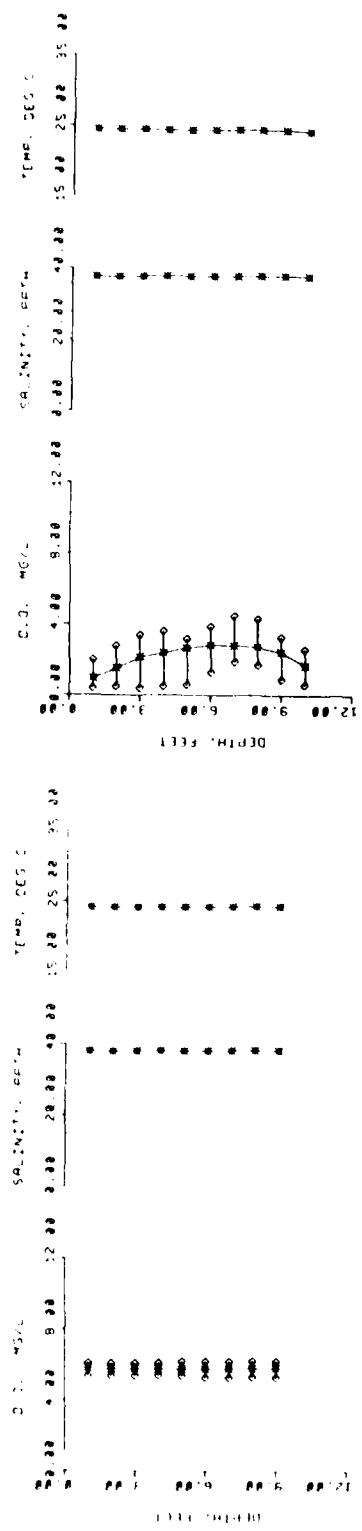


Figure II.9. Examples of D0 Profiles Above and Below Standards (4 mg/l) in Big Pine Keys Canals, Florida (from EPA, 1975)

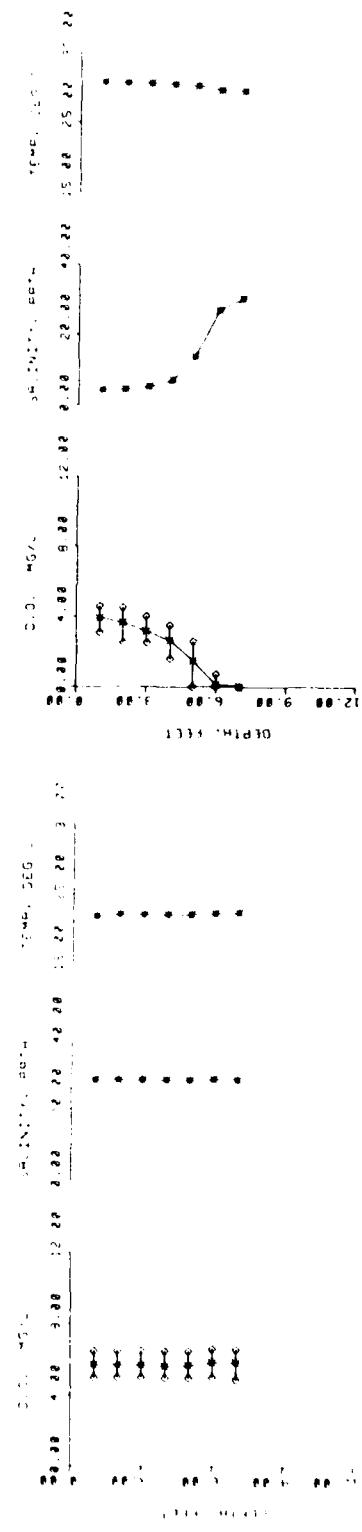


Figure II.10. Examples of D0 Profiles Above and Below Standards (4 mg/l) in Punta Gorda Canals, Florida (from EPA, 1975)

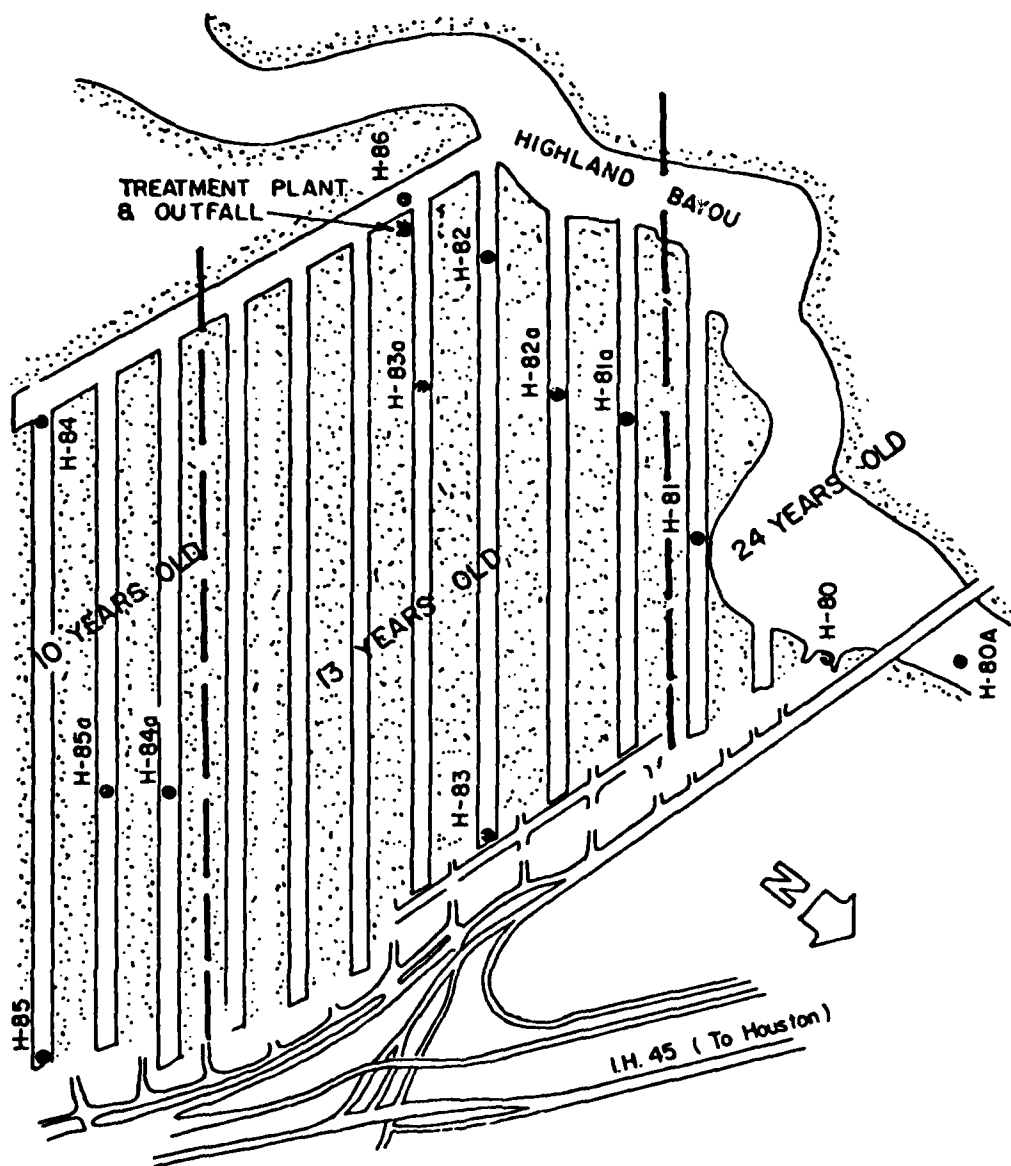


Figure II.11. Bayou Vista Canal System Showing Three Stages of Development (from Messenger and Reynolds, 1979)

of the canals studied in the system. Their results (Figure II.12) indicated that age was not a dominant factor in determining canal water quality when flushing and circulation are also exerting an effect. They believed this to be the case in this system, where the older canals are built closer to the receiving water body, and generally have better flushing properties.

Two physical features that have received much attention are canal depth and the presence of a sill. When a canal is first dredged, before connection to a receiving water body, a plug is often left in place. Upon removal, a sill may remain which impedes the circulation of the bottom waters. As a result, sediment may be trapped.

It has been observed that "deep" canals are not adequately flushed by tidal action, and that lower layers act as a trap for sediments and organic detritus. Polis (1974) and Barada and Partington (1972) reported thermal stratification in canals deeper than 15 feet. A sharp interface was measured between 10-12 feet, with indications of less turbidity, anaerobic conditions, and the presence of hydrogen sulfide below the interface. It has also been observed that canals that are very shallow (under 5 feet) may have poor flushing characteristics, poor navigability, and increased turbidity due to boat traffic (Chesher, 1974).

Sedimentation has been observed in many tidal canals. Such buildup may be caused by direct stormwater discharge into these systems, or high sediment loads from the receiving waters. Sediment buildup is usually associated with topographic features such as deep holes, sills, and bends, all of which tend to slow the water. On one occasion, a navigation project on the Intracoastal Waterway in Florida caused a side canal to silt up at its mouth and be impassible at low tide (Walton et al., 1975a).

The hydrodynamics and mass transport properties of tidal canal networks are closely interrelated. A thorough knowledge of these processes is needed to understand how to build these systems to reduce water quality problems and

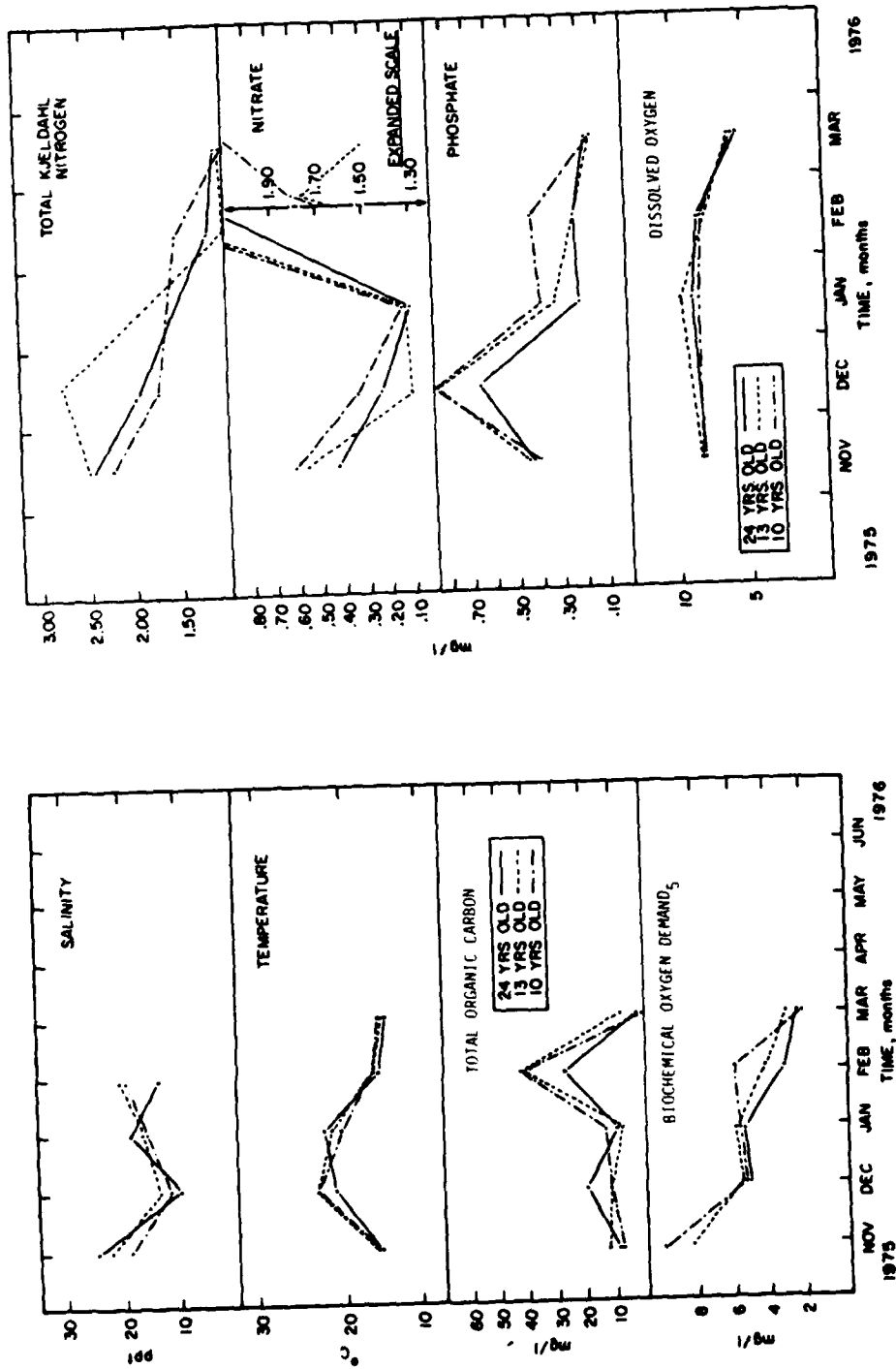


Figure II.12. Comparison of Water Quality Parameters in Canals of Various Ages (from Messenger and Reynolds, 1979)

to improve flushing and turnover where needed. A knowledge of these processes is needed if we are to be able to develop good predictive tools for canal design and evaluation.

II.2 Marinas

Many of the physical processes observed in canal systems are important in marina hydrodynamics and mass transport. Generally, however, time scales may differ due to a combination of geometry and tidal ranges, and processes may have a different order of importance. Typical geometric parameters are shown in Table II.2.

Marinas are usually built adjacent to large water bodies, such as estuaries and oceans, to provide protection and calm water for boat launching and storage. The circulation between the marina and the adjacent water body is a combination of tidal, density, and current effects. These processes are described by Vollmers (1976), Westrich (1976), Askren (1979), and Nece et al. (1976) and are shown in Figures II.13 and II.14.

Tidal ranges associated with marina locations are much more varied than for most canal systems. They vary from the low energy systems found along the lower Atlantic and Gulf Coasts to the relatively larger ranges found along the Pacific Coast and in New England. Ranges in the Puget Sound marinas study by Nece et al. (1972, 1974, 1975), for example, were between 3 and 13 feet depending on location. These relatively larger tidal ranges, together with the same order of low water depths for navigation, mean that flushing times are usually smaller (Figure II.15).

The current effect (Figure II.13) is not often noted in canal systems because they are relatively narrow. However, in many marinas there may exist a velocity shear between the basin and the adjacent water body which may produce one or more circulation cells (vortices - Figure II.16). The theoretical formation of such a circulation cell is described by Askren (1979)

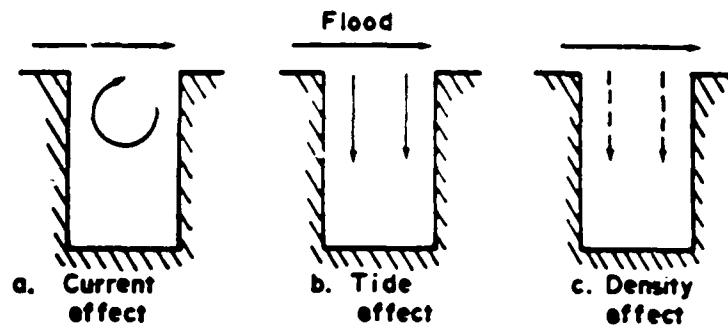


Figure II.13. Components of Marina Circulation
(after Vollmers, 1976)

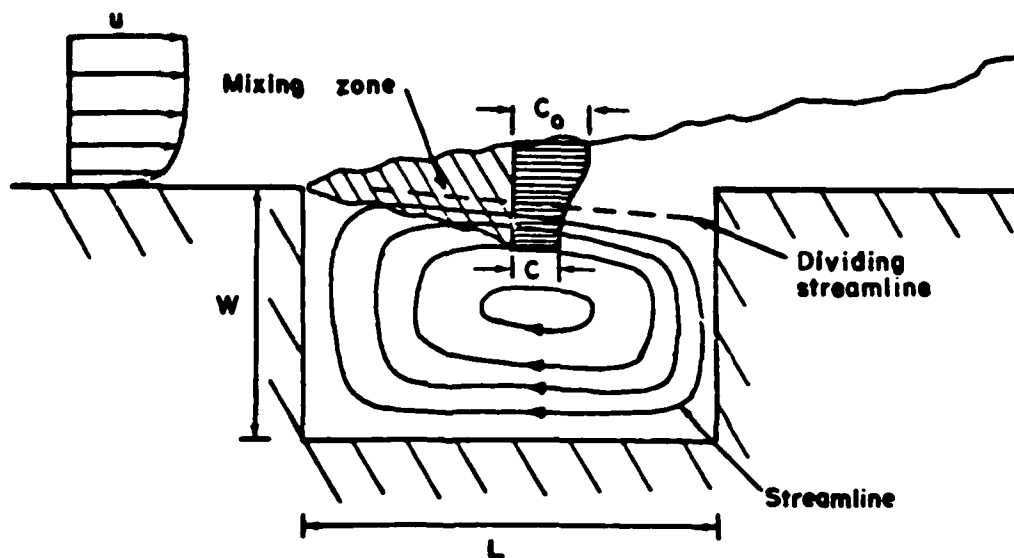


Figure II.14. Induced Circulation Pattern in Side Basin
(from Askren, 1979; and Westrich, 1976)

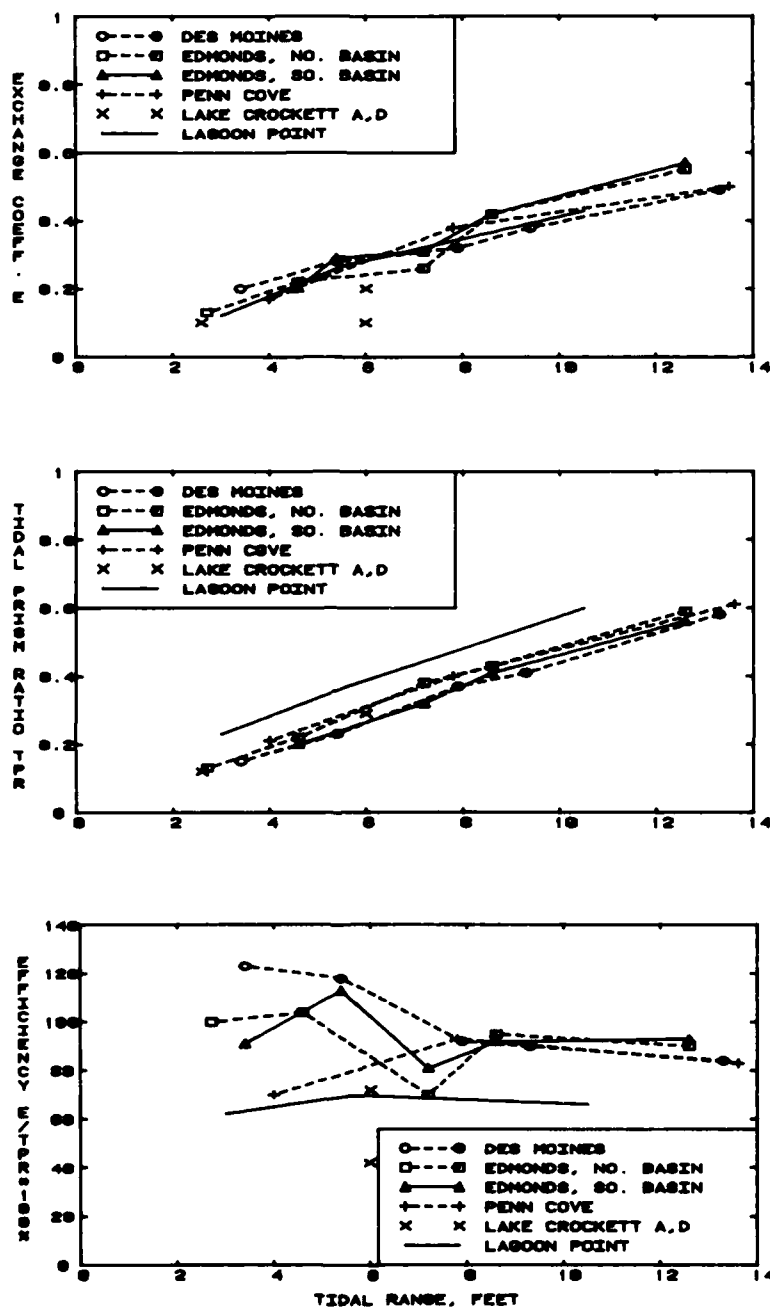


Figure II.15. Flushing Characteristics of Various Puget Sound Marinas (from Nece et al., 1975)

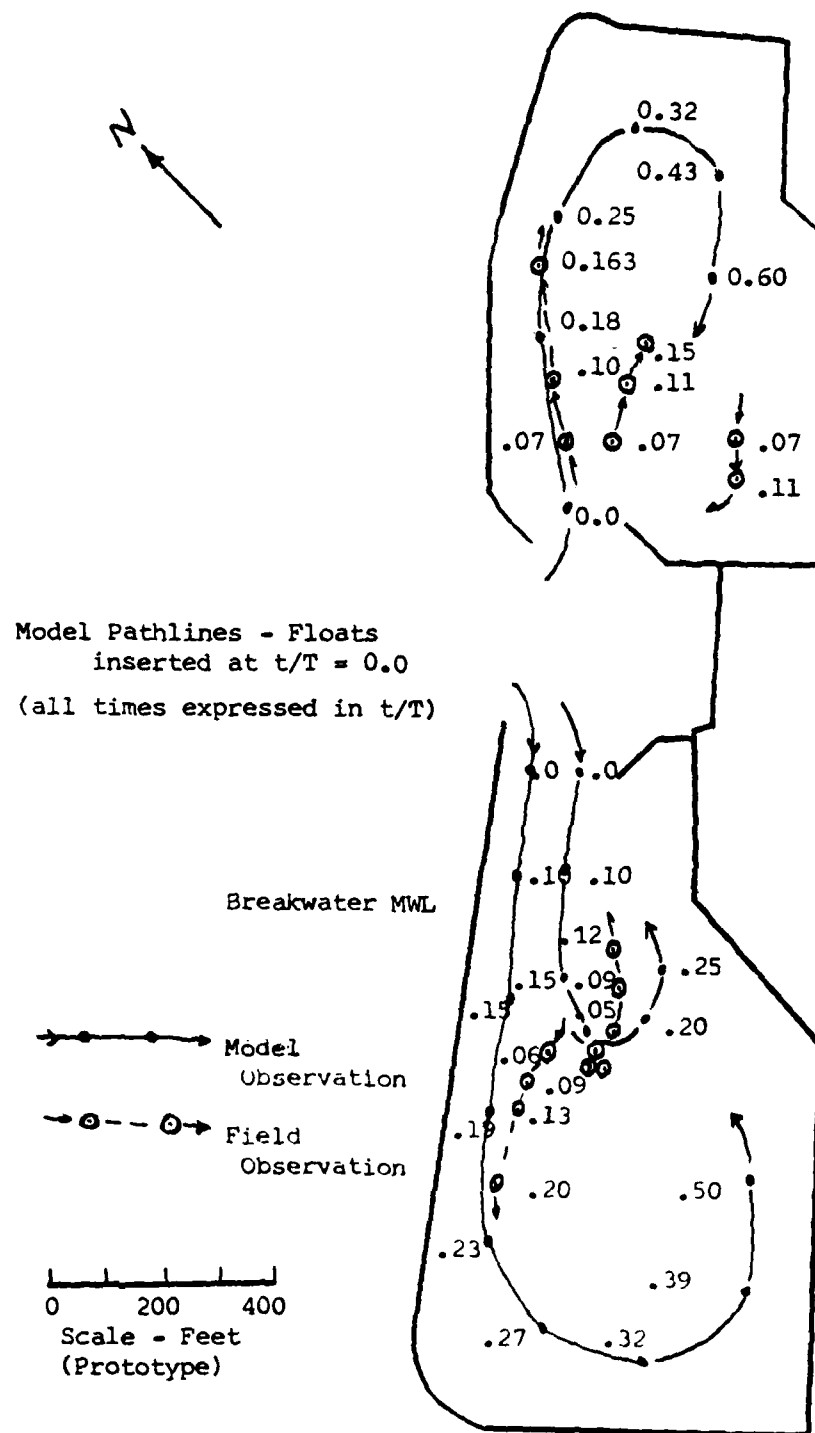


Figure II.16. Drogue Pathlines in Edmonds Marina
(from Nece and Knoll, 1974)

and Westrich (1976), and is shown in Figure II.14. Such cells can have an effect on water quality and sedimentation within the marina, but at this point their physics are not well understood.

Wind effects over marinas tend to be relatively unimportant, except in driving the adjacent water body. Over the marina itself, fetch lengths are small, and there is generally much masking due to trees and houses. Density-induced circulations are also small, as little fresh water enters these systems. Marina water may tend to be slightly warmer than that of the adjacent water body (Askren, 1979), but generally not sufficiently so to generate significant currents and changes in flushing rates in systems where tidal exchange dominates (Nece et al., 1979).

As marina development has grown, researchers are noticing more and more that the design which allows for calm, protected waters also acts as a sediment and pollutant trap. Ecologic and water quality studies are relatively few (Bowerman and Chen, 1971; Nece and Knoll, 1974; Nece et al., 1975), but results indicate that longer detention times can impair water quality (Figure II.17). In studies in West Coast marinas, Slotta and Noble (1977), Schluchter and Slotta (1978), and Swartz* have noted large amounts of benthic deposition, perhaps up to 2 meters. There is also some evidence that toxins may be building up in these systems, but this is a subject of ongoing research.** Ironically, it appears that benthic deposition may not improve with "good" flushing, but that it may actually increase as more sediment-laden waters are interchanged with the marina basin. Further studies have also shown that boat props may tend to resuspend benthic material, particularly in shallow marinas, and move materials around the basin. Such findings are preliminary, and are the first interpretations of long-term ongoing research into these and related processes.

* Personal Communication, R. Swartz, EPA, Corallis, OR, 1982.

** Personal Communication, L.S. Slotta, Oregon State University, Corvallis, OR, 1982.

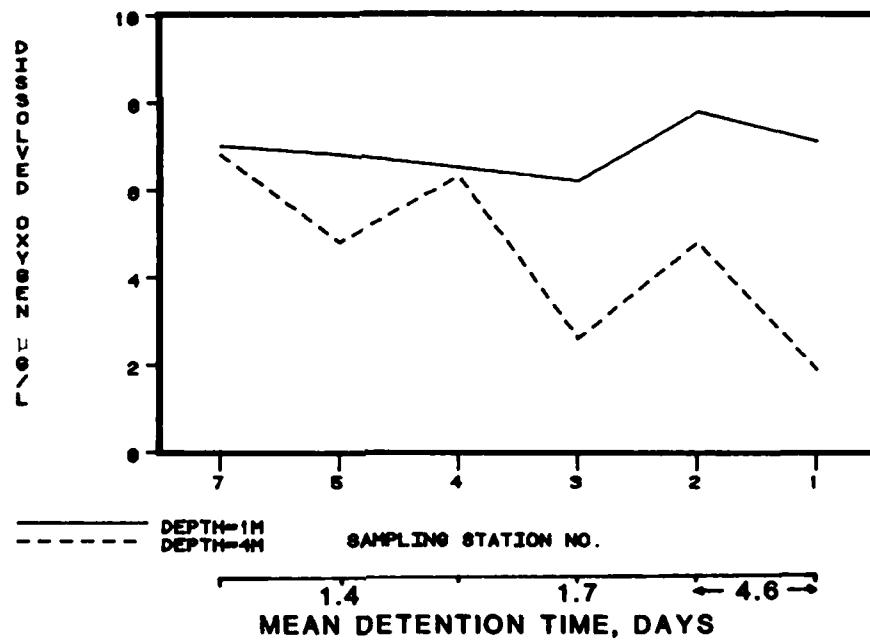
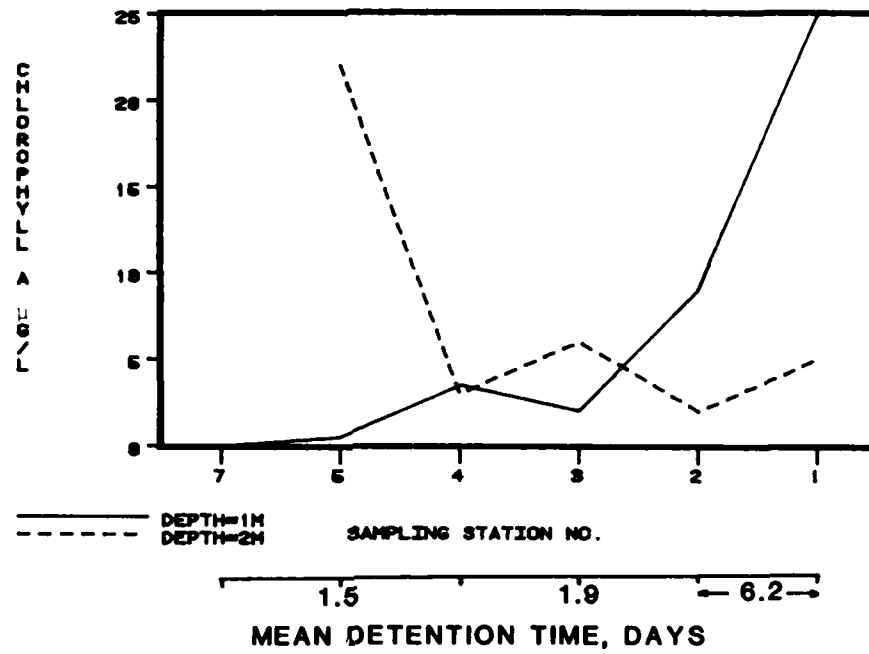


Figure II.17. Observations of Water Quality Parameters in Lagoon Point Marina (Nece et al., 1975)

An engineering approach to overcoming sediment buildup in some marinas on the East Coast is to design the basin with a lock structure. The object is to reduce the exchange of sediment-laden water, and also to reduce tidal fluctuations.* If this is done in combination with good engineering practice (van de Kreeke and Larsen, 1981), such as limiting pollutant loadings to the basins like stormwater runoff and allowing flushing using an upstream water supply, good water quality and low sedimentation rates can result.

* Personal Communication, B. Edge, Cubit Engineering, Inc., Clemson, SC, 1982.

III. DESIGN CONSIDERATIONS

Canal and marina design and construction must be accomplished within the limits of local, State, and Federal regulations. It is not our purpose to review these here, as regulations vary widely between states and even within states, but instead to discuss design considerations in the context of analytical evaluation of proposed systems.

Design standards and environmental regulations have tended to develop through a process of "learning by our mistakes." In some cases, reaction to these "mistakes" has been to swing the pendulum the other way and deny further permitting. This is somewhat the case for residential canal development in the State of Florida.

This process was examined by Snyder (1976a, 1976b) for canal developments in Florida. Early engineering criteria were (Figure III.1):

1. Navigable depths to the shoreline,
2. Maximized front footage per acre,
3. Increased elevation for foundations,
4. Minimum loss of property to water area,
5. Rapid drainage of rainfall, and
6. Simplified surveying and construction methods.

These criteria resulted from a combination of trying to maximize profits from a development, and applying hydraulic practices that dated back a number of centuries. In many cases, the criteria were borrowed from other types of development and caused problems in tidal canals.

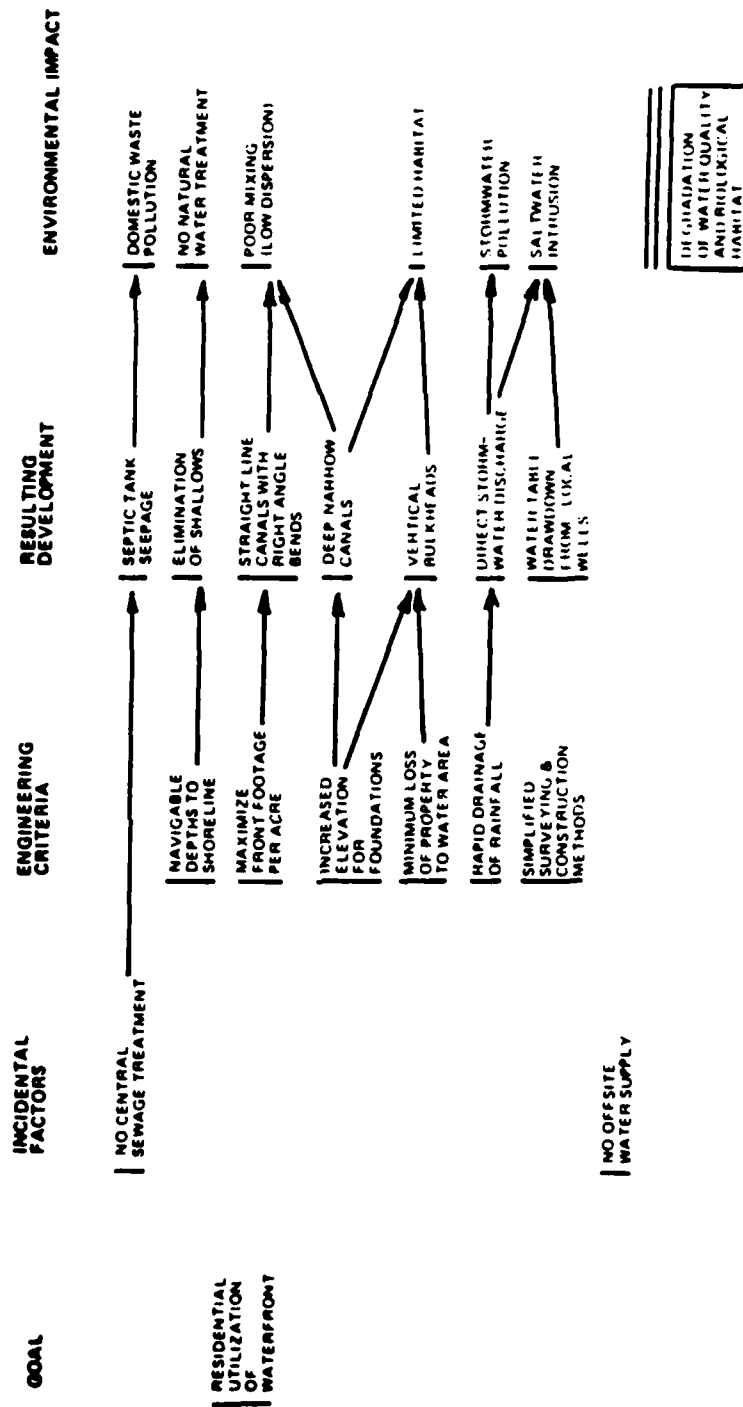


Figure III.1. Early Canal Design (from Snyder, 1976a)

Some of the results of the above approach were:

1. Septic tank seepage,
2. Elimination of shallows,
3. Straight canals with right angle bends,
4. Deep narrow canals,
5. Vertical bulkheads,
6. Direct stormwater discharge, and
7. Water table drawdown from local wells.

The environmental impact of canal developments using the above criteria are readily visible or measurable. They include domestic waste pollution, the lack of natural water treatment, poor mixing, limited fish and animal habitats, stormwater pollution, and saltwater intrusion. Because of the resulting environmental degradation, most regulatory agencies prohibit the construction of any new residential canals until it can be shown that such systems are compatible with the site, that the environment is not degraded, that all regulatory criteria are met, and that the system is rationally designed.

The regulatory agencies' response to the problems was to impose criteria and guidelines that they believed would promote more environmentally acceptable designs. Thus, EPA (1975) recommended that "an appropriate canal depth for shallow draft pleasure craft should be no more than four to six feet below mean low water," based on measurements of vertical dissolved oxygen profiles and numerical flushing models (Figures II.7 and II.8). Other criteria specify the minimum or maximum concentrations of given water quality parameters.

Such criteria and recommendations were a response to the problems, and some were developed through research and observations. Some make sense in environmental terms, and others need interpretation based on local conditions and other factors. As an example, Horowitz and Bazel (1977) analyzed the

process by which an advanced wastewater treatment plant was proposed for a community in West Florida. They found that the justification was to reduce phosphate levels below regulatory standards. It turned out that the ambient water was naturally higher in phosphates than the water to be treated.

This is not an attempt to criticize, but more a reminder that there is much to learn about physical processes and local affects before a rational set of design criteria can be imposed. That position will be reached by trial and error, and by research. For example, the EPA Toxic Standards for fresh water are now being revised (Wu et al., 1982) because in some areas the animals used to set the standard do not exist. In canal design this process of learning is leading to a new approach to canal development (Snyder, 1976a, 1976b; Messenger and Reynolds, 1979). Snyder, for example, proposed a new set of design criteria (Figure III.2):

1. Offsite sewage treatment or package plant,
2. Offsite water supply,
3. Retention of some shallows,
4. Habitat diversity--a combination of vegetated intertidal shallows, sloping rip-rap, and vertical bulkheads,
5. Increase circulation,
6. Promote mixing, and
7. Eliminate scour for specified conditions.

These are not a rigid set of criteria, but suggestions that should lead to more environmentally acceptable designs and compliance with existing regulations. Many of them are being adopted, and many are being further investigated. Van de Kreeke and Larsen (1981) and Snyder (1975), for example, discuss water quality standards for marinas, many of which are similar to those for canals and have arisen because of the same concerns. However, in many cases, the engineer faces the task of selling his design by showing that it will not degrade the environment, an unenviable task when there is so much still to learn about what constitutes an acceptable design.

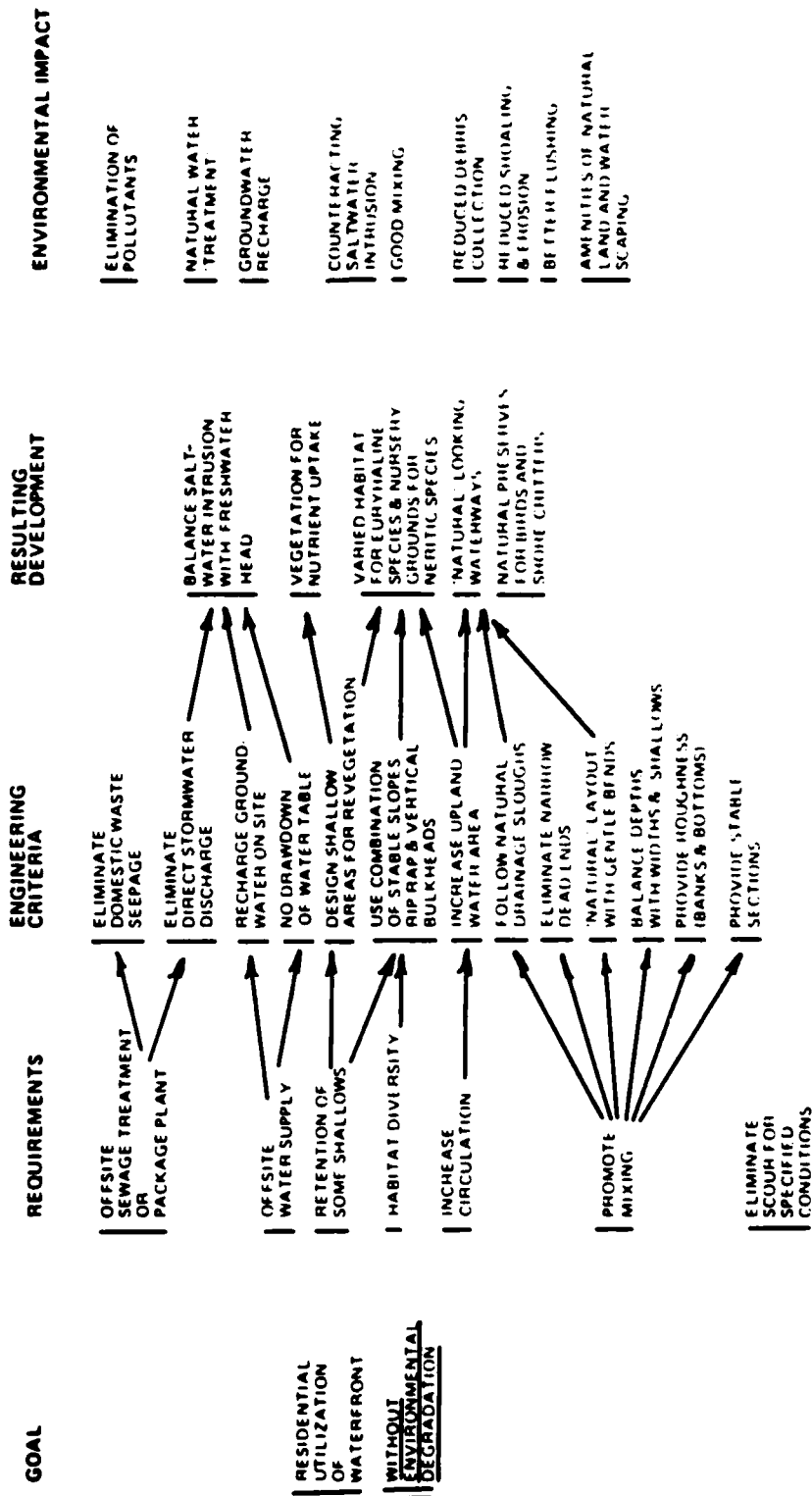


Figure III.2. Rational Approach to Canal Design (from Snyder, 1976a)

As a result, the design engineer must select an evaluation tool that is adequate to simulate the problem being investigated. The model must incorporate the appropriate physical processes, and produce results that a permitting agency will review for design acceptability. It must also include features that are elements of a "rational" design.

If, for example, wind is an important process in a dead-end canal system, the model selected should be capable of simulating multilayer flows in the vertical direction, as this process cannot be simulated using the area-mean velocity of a one-dimensional model.

Permitting agencies often want to know "how well does a system flush?" or "what will be the minimum DO concentration in the system?" to determine whether a proposed system will conform to regulations. They may wish to know what the sedimentation rates may be, and whether the banks will be stable. These processes should be included in the model.

Finally, a "good" design may include the use of multiple openings; flow-through systems; large upstream water bodies, such as lakes, to flush the system, etc. The ability to include such features must be built into the model.

In some cases the combination of existing processes and design criteria may simplify the modeling approach. For example, in South Florida canals, wind is known to drive circulation, but in design it may be appropriate to neglect its effect to determine whether a "worst case," or "poor situation" meets design standards.* In such a case, therefore, it would be appropriate to use a numerical model without a wind shear term to make this evaluation.

One of the roles of numerical models is the establishment of rational design criteria through model sensitivity. A model developed by Walton (1978)

* Personal Communication, J. van de Kreeke, Rosenstiel School of Marine and Atmospheric Science, University of Miami, Coral Gables, FL, 1982.

and Walton and Christensen (1980) was used by Morris (1978), Morris et al. (1978a, 1978b), and Hayes (1982) to investigate the effect of canal geometries on flushing a given solute distribution or inflow. The model simulates the solute transport process, and allows the design engineer to study transport and hydrodynamic features of a proposed or existing system.

Looking at tidal forcing only, Figure III.3 shows the effects of increasing channel length and inflow rates. Figure III.4 looks at the effect of adding an upstream lake to improve flushing in an idealized system by increasing the tidal basin volume. Such studies can give a feel for system response, but Hayes (1982) tried to further rationalize the process by developing a series of design curves that could be used instead of the computer program.

He performed a dimensional analysis to determine a series of dimensionless numbers appropriate for canal hydrodynamics and solute transport processes. By performing a series of simulations as variations of a standard canal (Figure III.5), he developed a parametric formulation of flushing:

$$c = M(R,S)M_1M_2M_3M_4M_5M_6 \quad (III.1)$$

where: c = concentration at tidal entrance,
 $M(R,S)$ = distribution of standard canal, and
 M_1-M_6 = functions of dimensionless numbers.

$$t_{\max} = D(R,S)D_1D_2D_3D_4D_5D_6 \quad (III.2)$$

where: t_{\max} = time to maximum concentration at tidal entrance (T),
 $D(R,S)$ = time to maximum of standard canal (T), and
 D_1-D_6 = functions of dimensionless numbers.

and

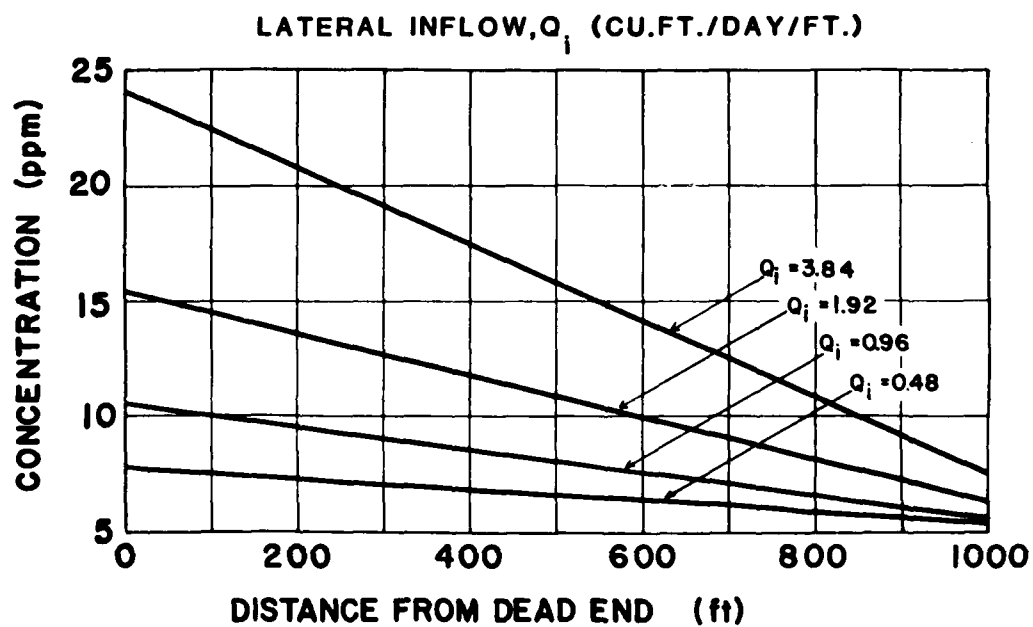
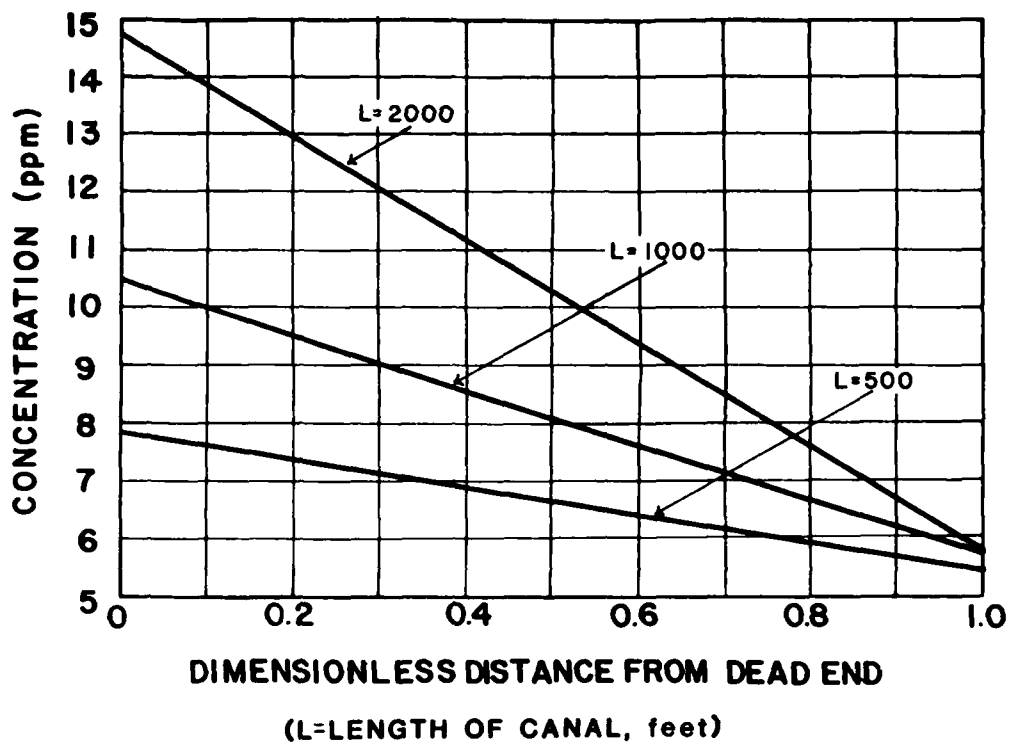
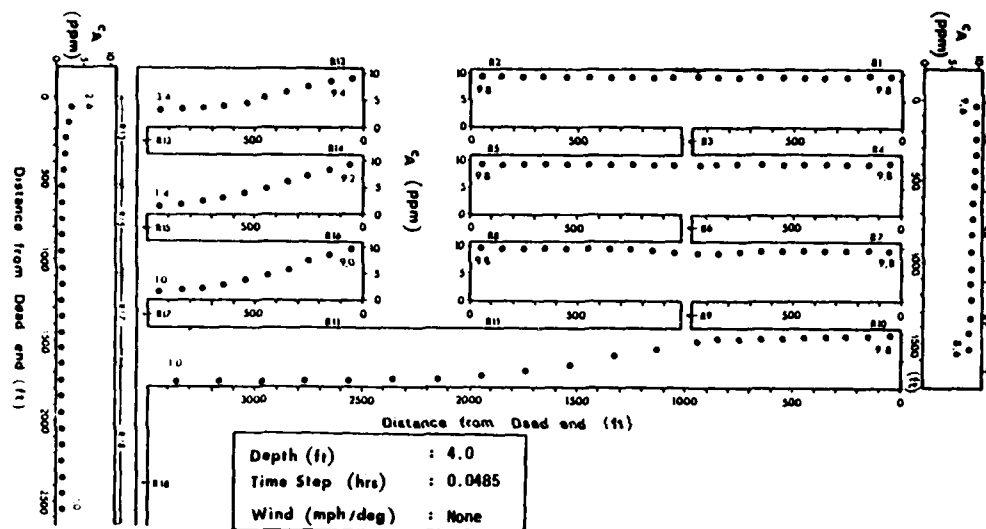


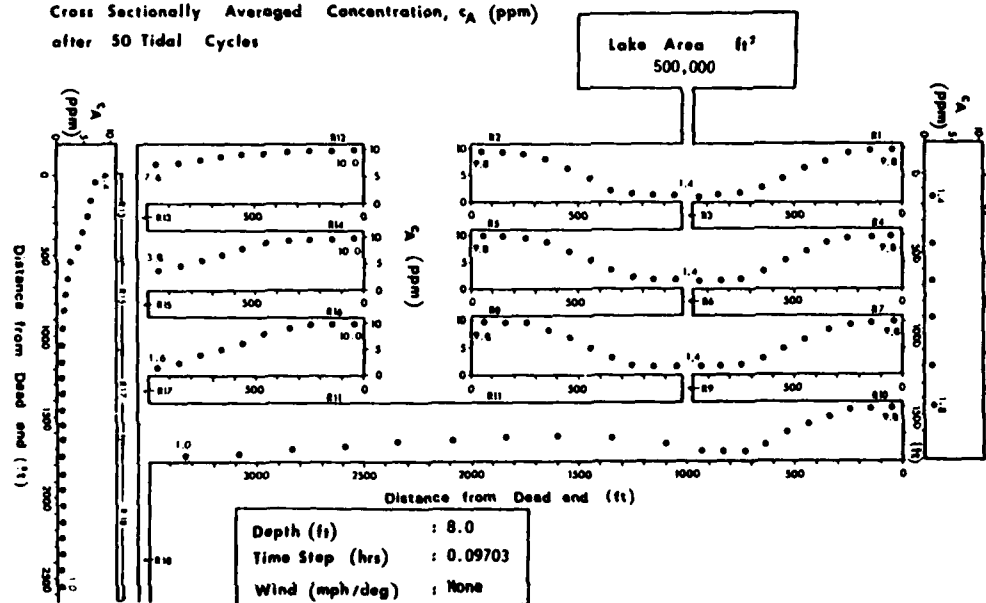
Figure III.3. One-Dimensional Model Sensitivity Analysis
(from Morris et al., 1978a)

Cross Sectionally Averaged Concentration, c_A (ppm)
after 50 Tidal Cycles



a. Without

Cross Sectionally Averaged Concentration, c_A (ppm)
after 50 Tidal Cycles



b. With

Figure III.4. Example of Model Sensitivity for a Canal System
Without and With a Lake (from Morris et al., 1978a)

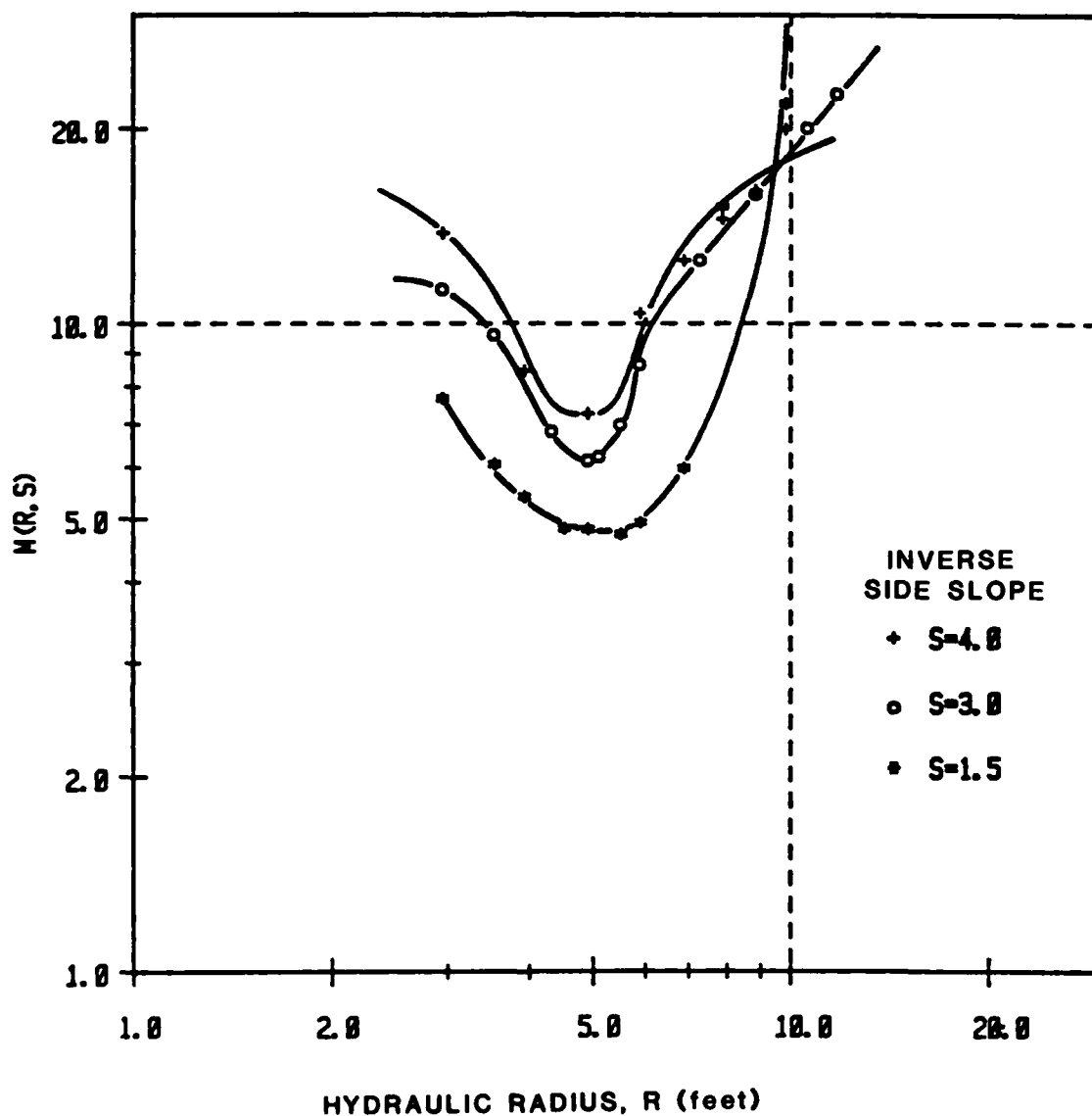


Figure III.5. Standard Runs to Determine Concentrations at Tidal Entrance of a Canal System (from Hayes, 1982)

$$t_{10} = N(R,S)N_1N_2N_3N_4N_5N_6 \quad (\text{III.3})$$

where: t_{10} = time to flush 90% of solute (T),
 $N(R,S)$ = time to flush 90% of standard canal (T), and
 N_1-N_6 = functions of dimensionless numbers.

Sensitivity analyses have been performed on marinas, but, to date, such studies have used either physical models or field measurements. Nece et al. (1975, 1976, 1979) used physical models to study geometric effects of marina designs. They investigated:

1. Planform geometry aspect ratio,
2. Ratio of entrance cross-sectional area to basin planform area,
3. Effects of rounding corners in basin interior,
4. Orientation and location of single entrances, and
5. Effect of two entrances versus a single entrance,

The results of the model dye studies were presented in terms of contours of equal, local, per-cycle exchange coefficients, E (Figure III.6), and average tidal flushing (Figure III.7) to develop some recommendations for good marina flushing.

Slotta and Noble (1977) developed a statistical model based on observations of sediments in existing marinas to determine flushing potential as a function of geometric properties. From the study they developed a nomogram (Figure III.8) for use in the design of marina sitings.

Such studies, using numerical models, physical models, and field measurements, are a sample of the work needed to develop rational design criteria. In the future, numerical models, as they perhaps become more representative of the physical and water quality processes existing in these systems, can be used to simulate many design alternatives easily and quickly to learn

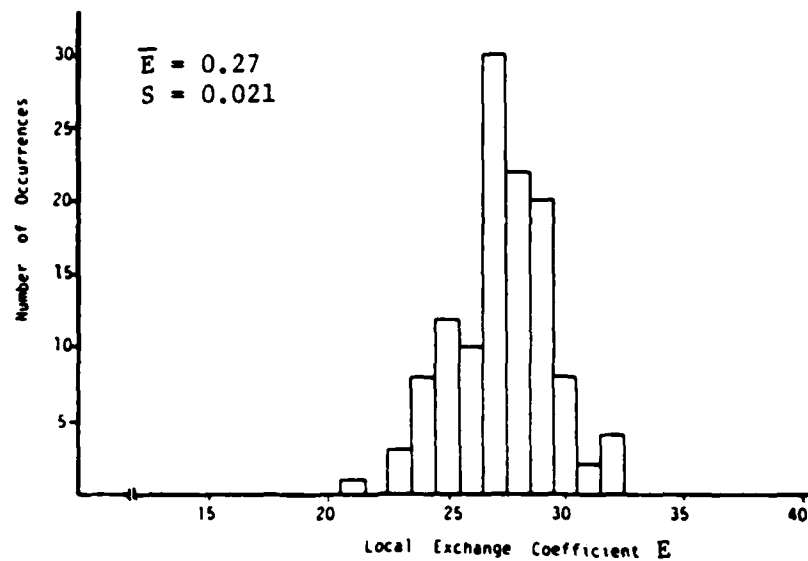
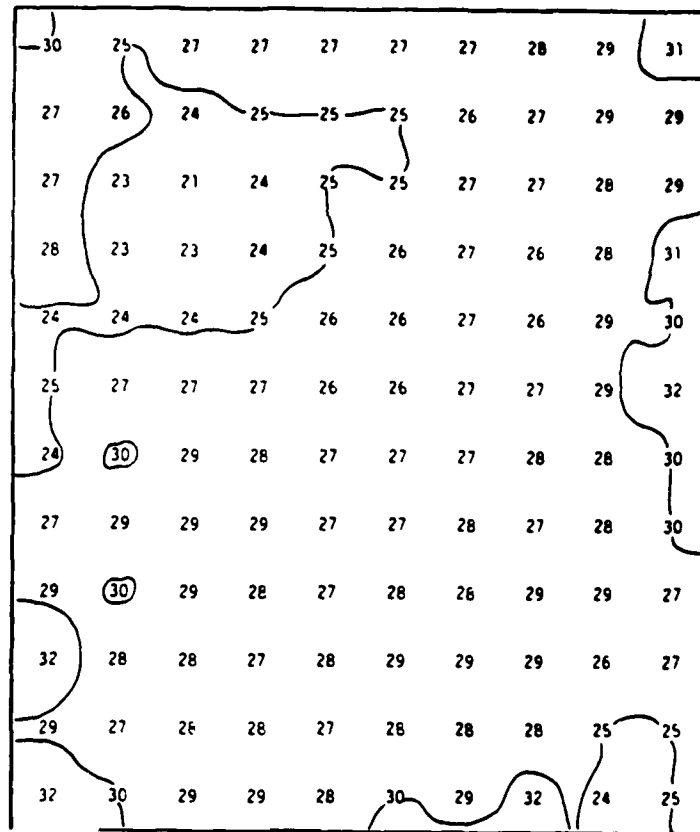


Figure III.6. Results of Test Case, Marina with Square Corners and a Single Asymmetric Entrance (from Nece et al., 1979)

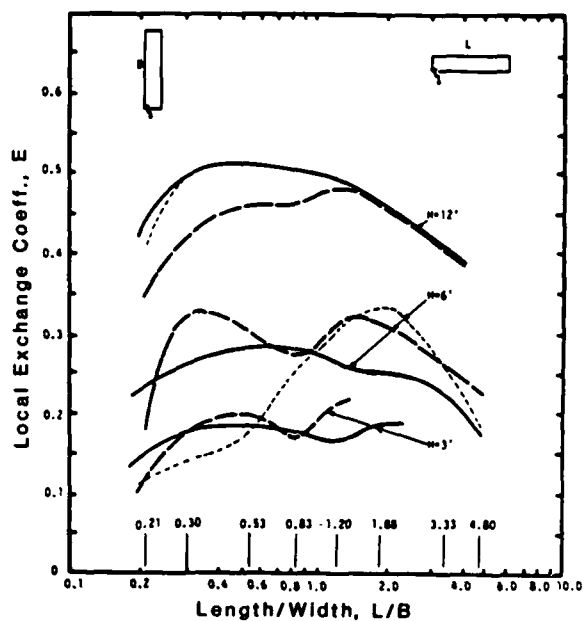


Figure III.7. Sensitivity of Marina Flushing to Geometry and Depth for a Single Asymmetric Entrance (Nece et al., 1979)

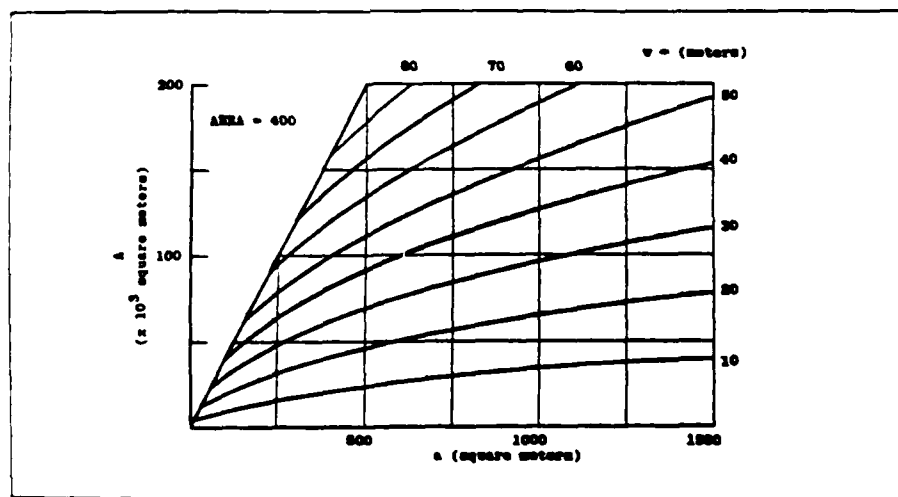


Figure III.8. Nomogram of Acceptable Marina Goemetry as a Function of Planform Area (A), Entrance Width (w), and Entrance Area (a) (from Slotta and Noble, 1977)

how such systems respond. At that point, they will be useful tools to design, evaluate, and establish criteria for environmentally acceptable systems.

To conclude, the selection of a model for evaluating proposed or existing systems must include the consideration of both physical processes and design regulations. The engineer must be aware of all the processes affecting an individual system before a decision can be made as to which ones to include in a model. This decision is further tempered by a consideration of what issues are being addressed (such as worst case analyses), what processes are important under those conditions, and what may be neglected.

IV. PAST STUDIES AND EXISTING MODELS

Having discussed canal and marina processes and design considerations, in this chapter we will review some past studies of canal and marina simulations, and examine the tools used for analytic evaluations. We will begin by considering some early designs before looking at techniques more commonly used today.

IV.1 Tidal Prism Analysis

Most of the early designs of canal and marina systems were based on a simple tidal flushing analysis. One of the earliest such studies was by Phelps and Velz (1933) to study pollution in New York Harbor. The tidal flushing analysis is based on volume exchange between the system and the adjacent water body. Assuming complete mixing at each exchange, and beginning at low tide, the flushing can be expressed in terms of the resulting concentration, c_N , after N tidal cycles:

$$c_N = c_0 \left(\frac{1}{1+F} \right)^N \quad (\text{IV.1})$$

where c_0 = initial concentration (dimensionless),

N = number of tidal cycles,

$F = V_p/V_L$,

V_p = tidal prism volume (L^3), and

V_L = volume of water in system at low tide (L^3).

or by considering the time to achieve a specified dilution, called the flushing time:

$$N = \frac{\ln (c_0/c_N)}{\ln (1 + F)} \quad (IV.2)$$

where N = number of cycles to desired flushing.

The above formulae assume a one-dimensional mean exchange between the system and adjacent waters, and that complete mixing occurs on each tidal cycle. This tends to overestimate flushing rates. Modifications were made to the above formulae by Gibson (1959) and Ketchum (1951) who divided the system into segments and assumed exchange between each segment to be completely mixed. A second modification was to assume some incomplete mixing of the tidal exchange volume. This rate would be determined empirically, often about 0.8, and would be included in Equations (IV.1) and (IV.2) by replacing F with αF , where α = fractional rate of exchange.

Even today, tidal prism analyses are used to evaluate flushing rates. Kuperman (1974) developed a segmented approximation to a canal system and empirically determined exchange coefficients. In studying South Beach Marina in Oregon, Callaway (1981) used a simple flushing model with a decay of concentration to simulate mixing during the flood tide. His results (Figure IV.1) show excellent agreement with a physical model of the system, but also show that the method can tend to be conservative when compared with field data. Such an approach, particularly for marinas, does appear to be a reasonable "back-of-the-envelope" calculation to obtain some idea of the exchange of water and solute mass between the marina and the adjacent waterway.

It is further interesting to note that such an approach can give a reasonable estimate of numerical model results of flushing. Lo et al. (1976) compared a simple tidal prism method with a one-dimensional network model and achieved quite similar results (Figure IV.2).

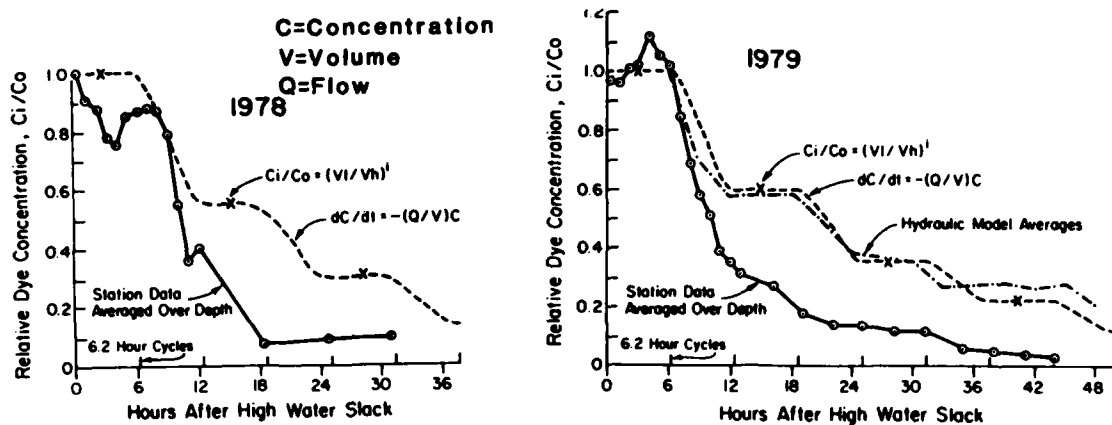


Figure IV.1. Tidal Prism Flushing Model of South Beach Marina
(from Callaway, 1981)

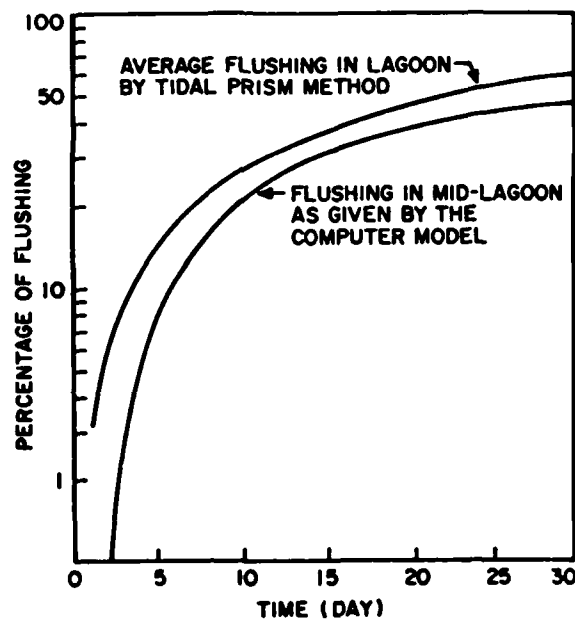


Figure IV.2. Comparison of Tidal Prism and Numerical Model of Flushing (from Lo et al., 1976)

IV.2 One-Dimensional Models

With the advent of computers, the design engineer was able to use more sophisticated methods to evaluate proposed systems. Instead of estimating flushing rates based on volume exchanges, hydrodynamic and mass transport processes could be directly simulated and results obtained within a relatively short period of time. The models allow the design engineer to look at not only flushing rates, but also to evaluate water quality conditions and circulation patterns.

The basic governing equations in one dimension are

the momentum equation:

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} (Qu) + \frac{A}{\rho} \frac{\partial P}{\partial x} + KB |u|u = B\phi + \frac{\partial}{\partial x} (N_L \frac{\partial u}{\partial x}) \quad (\text{IV.3})$$

the continuity equation:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_I \quad (\text{IV.4})$$

and the mass transport equation:

$$\frac{\partial}{\partial t} (Ac) + \frac{\partial}{\partial x} (Qc) = \frac{\partial}{\partial x} (AE_L \frac{\partial c}{\partial x}) - KAc + q_I c_I \quad (\text{IV.5})$$

where Q = flow (L^3/T),
 t = time (T),
 x = distance (L),
 u = area mean velocity (L/T),
 g = acceleration due to gravity (L/T^2),
 A = cross-sectional area (L^2),
 ρ = density (M/L^3),

P = pressure (M/LT^2),
 k = friction coefficient (dimensionless),
 B = top width (L),
 ϕ = wind shear (L^2/T^2),
 N_L = longitudinal momentum transfer coefficient (L^2/T),
 q_I = lateral inflow rate per unit length (L^2/T),
 c = concentration (dimensionless),
 E_L = longitudinal dispersion coefficient (L^2/T),
 K = decay rate ($1/T$), and
 c_I = lateral inflow concentration (dimensionless).

Most existing one-dimensional models are based on the above equations or simplified versions of them.

Michel (1973a, 1973b, 1973c) used a model with simplified versions of Equations (IV.3) and (IV.4) to study canal and marina circulation in Florida. He assumed that the velocity, u , could be given by Manning's equation in the form:

$$u = \frac{1.486}{n} R^{2/3} S^{1/2} \quad (IV.6)$$

where n = Manning's roughness coefficient ($T/L^{1/3}$),
 R = hydraulic radius (L), and
 S = hydraulic gradient (dimensionless).

Equation (IV.6) can be formed from Equation (IV.3) by assuming a balance between the pressure force and the frictional resistance. He used the model to look at the changes in flows resulting from proposed geometric changes due to marina development.

Another simplification of canal hydrodynamics involves the assumption of a horizontal water surface. As was noted in Section II.1, flows in canals exhibit very small water surface slopes and velocities. Tide gages placed at

either end of a system would, in many cases, show little difference in elevation. If we assume the water surface to be a horizontal plane, tidal velocities can be calculated as closed form functions of upstream geometry:

$$u = \frac{A_s}{A} \frac{dh}{dt} \quad (\text{IV.7})$$

where A_s = area of water surface (L^2), and
 h = tidal depth (L).

The above model (Walton, 1976a, 1976b; Walton and Christensen, 1977) was compared with a model of Harleman and Lee (1969), based on the full equations (IV.3) and (IV.4). The results never differed by more than five percent, even over relatively long canals. Such variations are well within error limits in measurements.

Knowing the velocity allows the longitudinal dispersion coefficient to be calculated as a closed form function, based on the formulations of Taylor (1953, 1954), Elder (1959), Walton (1978), and Walton and Christensen, (1980):

$$E_L = K_L hu + E_0 \quad (\text{IV.8})$$

where K_L = dimensionless longitudinal dispersion coefficient, and
 E_0 = background longitudinal dispersion coefficient (L^2/T).

Equations (IV.7) and (IV.8) form the basis of the hydrodynamic and solute transport models of Langley (1976), Lee (1977), Walton (1978), and Walton and Christensen (1980).

Westrich (1978) used a different approach for studying marina-type basins adjacent to an unsteady main stream current. By considering the physics of the mass transfer process, he developed a model that simulated mass transport in both the main channel and the basin, and empirically defined

exchange coefficients between them. The model performed well against experiments, but more work needs to be done to compare the results against field data and to determine the range and sensitivity of the exchange parameters.

In most cases, however, one-dimensional models are developed based on approximations to the full governing equations to simulate hydrodynamics, flushing, or water quality transport. Generally, hydrodynamic models were developed to look at flows through large canal systems, such as inter-oceanic canals, or by interbay canals (Keulegan, 1966; Sagar, 1969; Harleman and Lee, 1969; Boyd et al., 1973; Gardner and Pritchard, 1974; Johnson, 1974; van de Kreeke and Dean, 1975; Rives and Pritchard, 1978; Najarian et al., 1980). Such models were used to determine heads and velocities, and to estimate net flows.

When evaluating residential canals and marinas in the coastal zone, water quality, rather than flow, is usually the issue. In fact, as relatively quiescent systems are desired for navigation and boat storage, many of these systems have been found to exhibit poor flushing, and have water quality problems or excessive benthic deposition.

The most popular one-dimensional water quality model is based on the link-node concept developed by Water Resources Engineers in the mid-1960's (Shubinski et al., 1965; Feigner and Harris, 1970). This original model took two different directions, toward the Dynamic Estuary Model (DEM), and toward the RECEIV model. The formulation for each series of models, however, is very similar, and between them they have modeled more estuaries, canals, and marinas than any other model.

Recently, Jettmar et al. (1980) used a modified version of DEM to study a canal system in southeast Florida. Figure IV.3 shows the results of a comparison between computer results and field measurements. Morris et al. (1980) used a version of the model to study flushing properties in another south Florida canal.

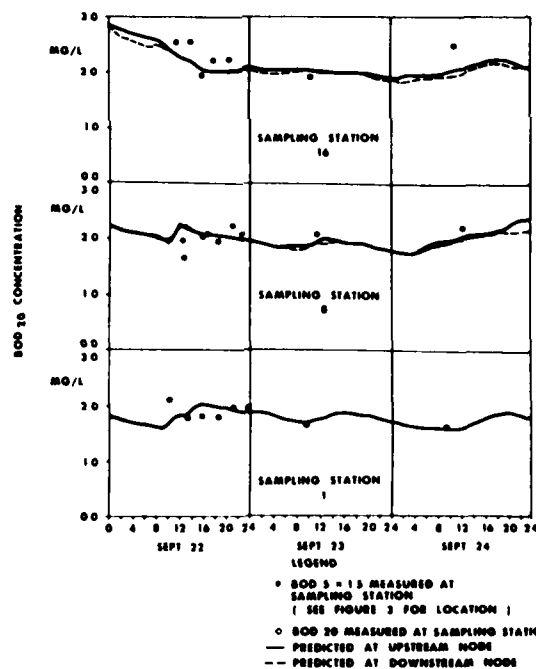
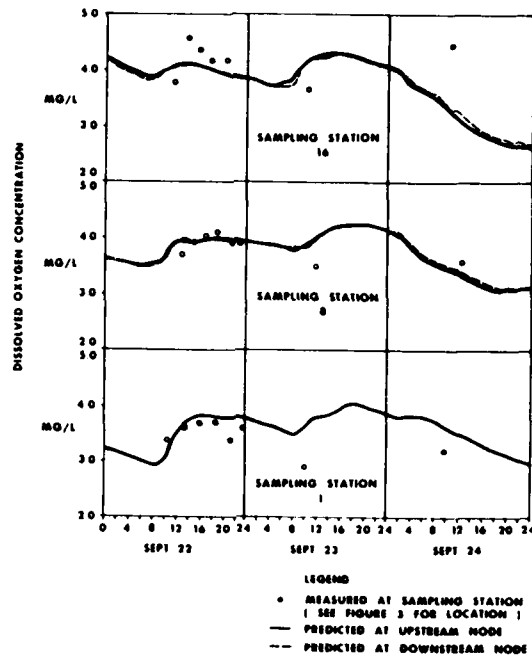


Figure IV.3. Water Quality Simulation in Southeast Florida Canal (from Jettmar et al., 1980)

Other water quality models have been used with varying degrees of success. Russo and McQuivey (1975) applied the simple water quality model, QUAL-1, to a south Florida canal. Figure IV.4 shows that they had some difficulty in obtaining of good fit between computed results and observations. Brandsma et al. (1973) and Lo et al. (1976) applied one-dimensional branching models to investigate mass transport and flushing in marinas.

Perhaps the most comprehensive investigation of water quality modeling in canals, however, was performed by EPA in the early 1970's (EPA, 1973, 1975; Barnwell and Cavinder, 1975). They studied a number of canal systems from Florida to North Carolina, performing dye, water quality, and modeling studies. More than any other study, this one allows us to investigate whether such an approach is appropriate for these systems.

Big Pine Key Canal is one of several canals in the Florida Keys studied by the EPA (1973, 1975). The canal (Figure IV.5) is almost 1,600 feet long, 40 feet wide, and 10 feet deep at mid-tide. It has a rectangular, prismatic cross section and is aligned with its longitudinal x-axis (defined as positive from the dead end) predominately NNW.

On 3 November 1973, at high water slack, 500 ml of Rhodamine WT dye was injected at mid-depth, 50 feet from the dead end of the canal. At regular intervals after the injection, dye concentrations were measured using a fluorometer sampling at mid-depth along the longitudinal center line of the canal.

Once the data were collected, EPA tried to simulate the profiles, using first a version of the Storm Water Management Model (SWMM) (Langer et al., 1971, Metcalf and Eddy, et al., 1971), and then the Columbia River Model (CRM) (Callaway et al., 1969; Callaway and Byram 1970). The reason given for turning away from the SWMM model was that (EPA, 1975):

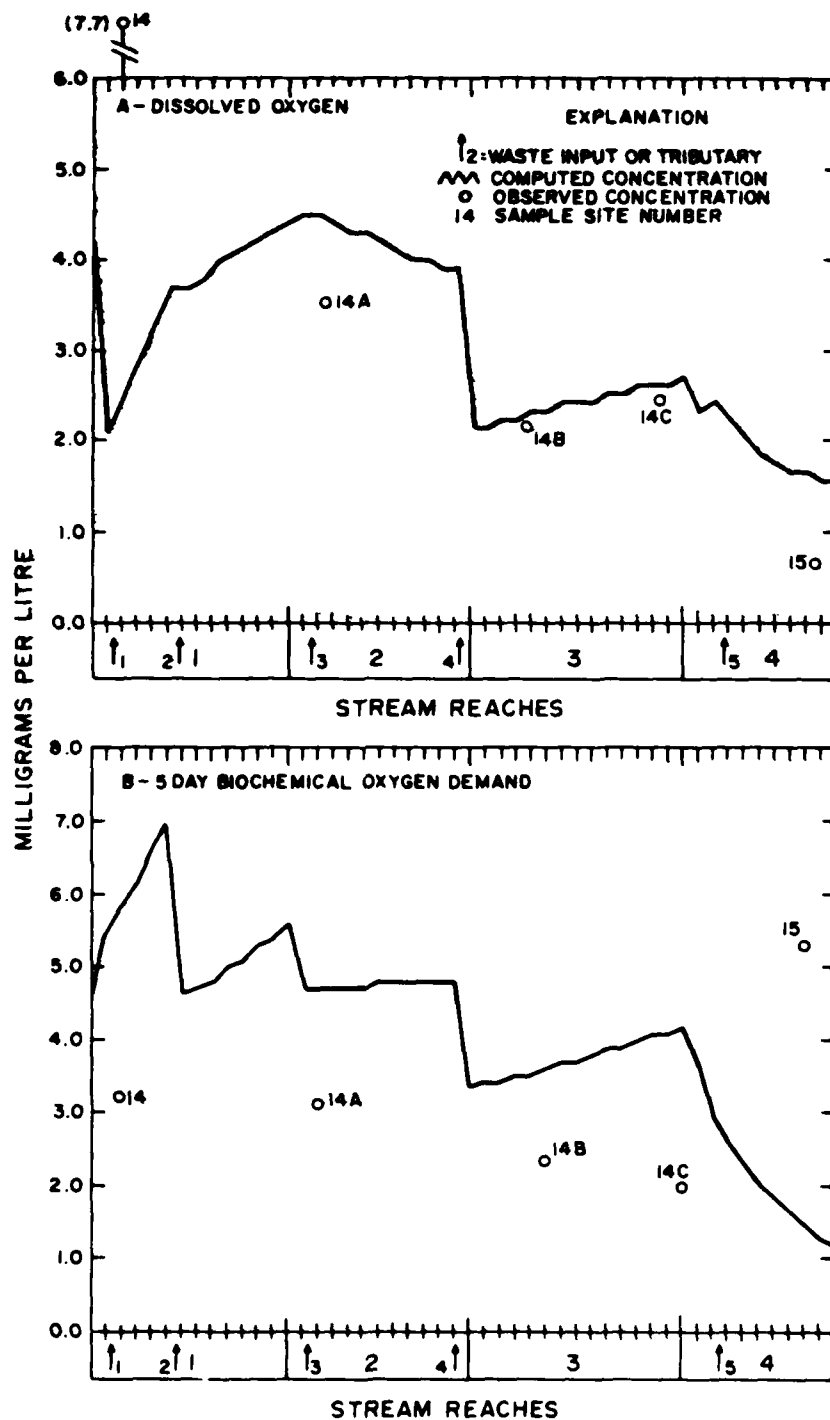


Figure IV.4. Comparison Between Computed Results and Observations (from Russo and McQuivey, 1975)

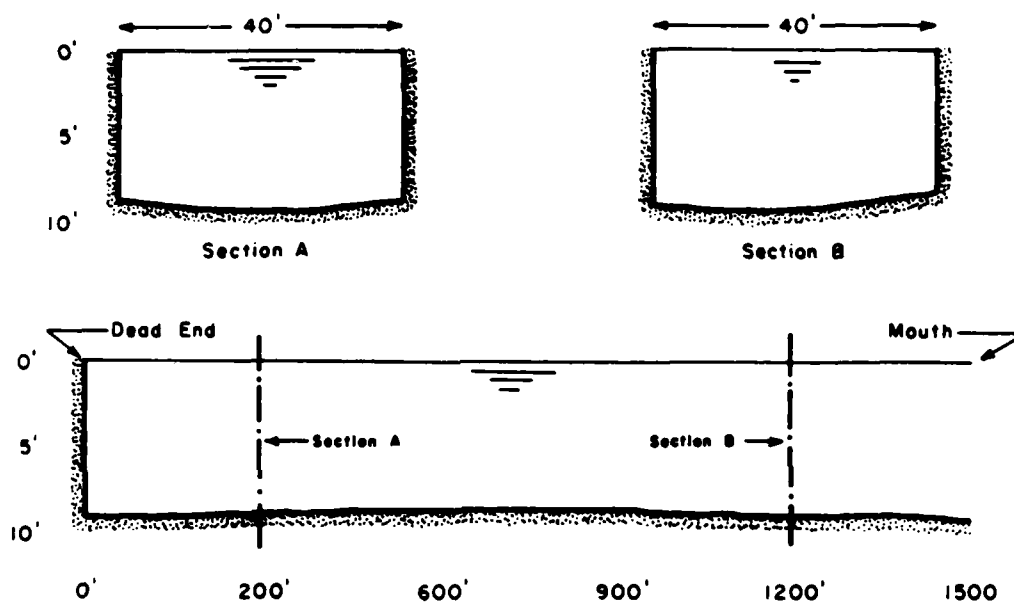


Figure IV.5. Longitudinal Section and Cross Section of Big Pine Key Canal (from EPA, 1973)

SWMM did not satisfactorily reproduce observed profiles. The field study showed the dye mixing rapidly throughout the length of the canal and then slowly "bleeding" out. The model, however, shows the dye cloud centroid remaining in one place and slowly flattening out, similar to classical diffusion without advection. As the SWMM neglects diffusion in its transport equation, this apparent diffusion is probably due to "numerical dispersion", or "pseudo-dispersion", associated with the numerical methods employed in this model.

Turning to CRM, which incorporates a dispersion term, several of the canals examined were modeled and the results presented. However, the measured profiles were reproduced by making an adjustment to the dispersion term, effectively dividing it by five.

Lee (1977) also tried to model Big Pine Key Canal using a one-dimensional model which corrects for numerical dispersion. However, in obtaining a fit between computed and observed profiles he had to increase the dispersion term by two orders of magnitude (Figure IV.6).

From these results, two conclusions may be drawn. Firstly, the CRM suffers from severe numerical dispersion when used to model low energy canals such as those along the Gulf and East Atlantic coasts. This is not really too surprising, as the CRM and SWMM are interrelated models, both having been developed from the Dynamic Estuary Model (DEM). The inclusion of a dispersive term in the CRM cannot be expected to produce any better results, as the main source of error has not been addressed.

Secondly, as stated in the EPA report (1975), "the dye cloud centroid remaining in one place" must be expected in a model which, due to its one-dimensional form, can only model the velocity field induced by the astronomical tide. In the Big Pine Key area of Florida, the tidal range is on the order of 1 foot. A simple volumetric tidal prism analysis indicates

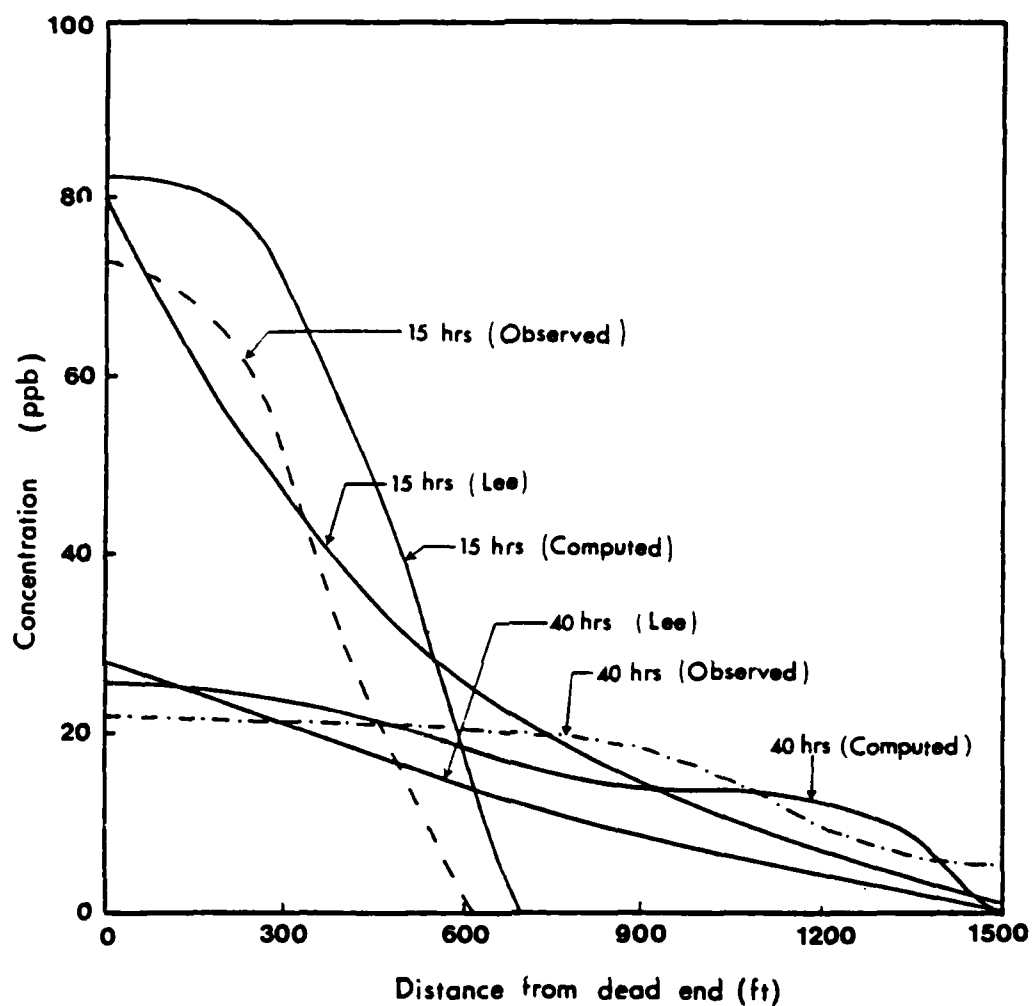


Figure IV.6. Comparison of Observations and Results of Models of Lee (1977) and Walton (1978) and Walton and Christensen (1980 - Computed) in Big Pine Key Canal

that maximum velocities of no greater than 0.01 foot per second can be expected, and then only at the tidal entrance. At the dead end of the canal, the tide-induced velocities will be much lower, and certainly not enough to move the dye cloud any appreciable distance.

In a system which is directly open to the ocean, and which does not appear to be influenced by density currents, the main forcing function producing circulation in the canal must be the wind. This effect cannot be modeled in one dimension, although many people apparently have tried. The altering of dispersion coefficients to match prototype conditions using an ill-defined model can only give results good for that particular case--and no other. Any predictive analysis based on such a model calibrated in this manner must be at best questionable.

To illustrate this point, the three-dimensional model CANNET3D (Walton 1978, Walton and Christensen, 1980) was run for this case. Three layers were used in the vertical and an estimated wind speed, from monthly averages at Key West, was imposed. As can be seen from Figure IV.6, a closer approximation to the shape of the distribution results, using reasonable values of transfer coefficients.

Part of the problem was that, in many instances in the past, one-dimensional water quality models, which were really intended for other systems, have been applied to canals and marinas. Specific problems were:

1. incomplete physics modeled,
2. incomplete processes modeled, and
3. models had excessive numerical dispersion.

As we have seen in Chapter II, canal systems in the vertical sense, and marinas in the horizontal sense, exhibit two-dimensional flows on many occasions. Attempts to study these systems using one-dimensional models have meant that major processes may have been ignored, and that their effects

had to be lumped into a group of uncertainties, called dispersion and momentum transfer coefficients. These unknown processes are then modeled using a gradient transfer mechanism that may not even approximate the ignored physics.*

A second problem is that many of the numerical models are not set up to accurately control numerical errors. When modeling the one-dimensional advection equation:

$$\frac{\partial c}{\partial t} + u \frac{\partial c}{\partial x} = 0 \quad (\text{IV.9})$$

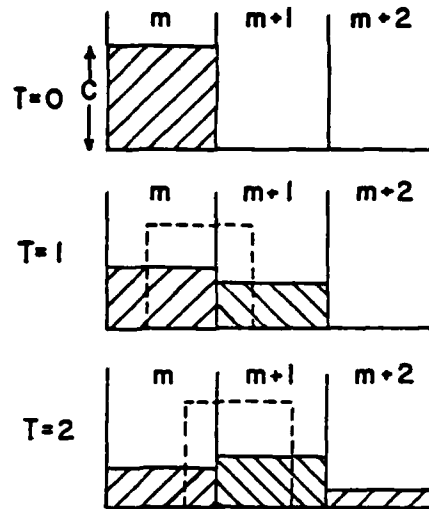
for example, using an explicit upwind difference scheme, we obtain a numerical dispersion term in the form (Molenkamp, 1968) $E_n \frac{\partial^2 c}{\partial x^2}$ where:

$$E_n = \frac{|u|\Delta x}{2} \left(1 - \frac{|u|\Delta t}{\Delta x}\right) \quad (\text{IV.10})$$

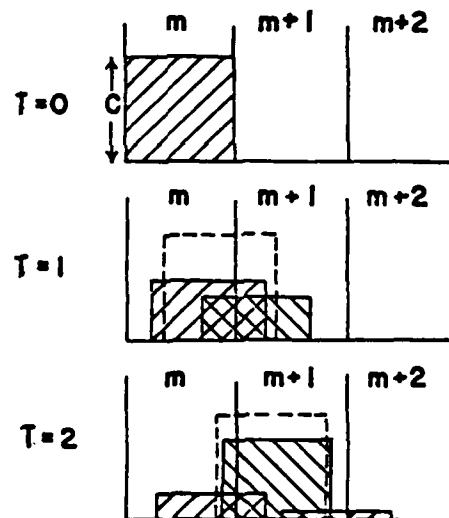
where E_n = numerical dispersion coefficient (L^2/T),
 Δx = distance interval (L), and
 Δt = time interval (T).

Figure IV.7a shows the effect of numerical dispersion for this scheme. If the center of mass is also included as a solution parameter, more accuracy is gained as shown in Figure IV.7b. Including the second moment, or width, allows the square wave to be advected without loss of shape. This latter scheme is the basis for the canal model of Walton (1978) and Walton and Christensen (1980). Other schemes, such as the central difference explicit, are known to be unstable. These schemes are discussed at length in Roache (1972).

* Personal Communication, J. van de Kreeke, Rosenstiel School of Marine and Atmospheric Science, University of Miami, Coral Gables, FL, 1982.



a. Upwind Difference



b. Upwind Difference with Conservation of Center of Mass

Figure IV.7. Numerical Damping

In modeling rivers, such a scheme may not produce too severe an error, or numerical damping, as longitudinal dispersion coefficients may be on the order of $100 \text{ ft}^2/\text{sec}$ or more, and may dominate the error. In canals and marinas where $\Delta x = 100 \text{ ft}$, $\Delta t = 100 \text{ sec}$ and $u = 0.5 \text{ ft/sec}$ is reasonable, $e_n = 12.5 \text{ ft}^2/\text{sec}$. The longitudinal dispersion coefficient, E_L , observed in these systems may be $5 \text{ ft}^2/\text{sec}$. Thus, a model simulation of such a system using the above conditions would produce severe numerical damping, and the results would be questionable.

The problem of severe numerical dispersion can be overcome to some extent by developing a scheme to minimize the error produced. The schemes of Lee (1977), Walton (1978), Walton and Christensen (1980), and Fischer (1970, 1972) specifically addressed this issue for canal systems. Other solution techniques, such as those of Hall and Chapman (1982), Leonard (1979), Fischer (1978), and Glass and Rodi (1982), could be incorporated into canal and marina models to reduce numerical errors in the mass transport solution.

We have shown in this section that one-dimensional models cannot always be used to describe and simulate the processes observed in canals and marinas. In some cases, such as flow-through systems or where the tide dominates, they may be adequate. In others, such as dead end systems or systems that exhibit two-dimensional, or more, flow, they may be less than adequate to simulate prototype processes. These considerations must be carefully weighed before selecting a modeling approach.

IV.3 Multidimensional Models

Multidimensional models for canal simulations are rare, and when they do exist, the second dimension modeled is the vertical. This is reasonable in most cases because canal systems are relatively narrow, and physical processes such as wind and density currents tend to produce vertical variability or flow reversals.

Such models have usually been developed to overcome the problems identified in the previous section. Monopolis (1978) realized that density-driven circulation could not be simulated well using a one-dimensional approach. To simulate the effect of freshwater inflows to saltwater canals in south Florida, he developed an analytic model, incorporating both the longitudinal and vertical directions, to predict velocities and salinities. He noted that the results (Figure IV.8) shown at the surface were similar in form to observations, but more work needs to be done to determine the utility of the model against field data.

Walton (1978), and Walton and Christensen (1980) developed a pseudo-three-dimensional model using a "flexing" cell structure. Using a horizontal water surface assumption, velocities and dispersion coefficients may be specified as closed-form functions of geometric properties using the continuity equation with head given at the tidal entrance. A method-of-second moments technique was used to minimize numerical dispersion, and the model was applied to several canal systems in the State of Florida.

The model generally performed well, and calibration parameters tended to be fairly uniform over widely varying systems. This led to the belief that, by including the correct physical processes, even in simplified form, a model could be developed that was fairly independent of calibration conditions, and this might be useful as a design tool. Comparisons with field data showed that the simplified velocity field corresponded satisfactorily with velocities in a wind-driven system (Figure IV.9) and in a density-driven system (Figure IV.10). Finally, dye studies were simulated, and again the results (Figure IV.11) showed good agreement, although much room for improvement remains.

The model does, however, have some drawbacks that limit its usefulness. It was developed to model upward branching systems and cannot handle multi-entrance or flow-through systems. Unfortunately, designers now realize that many of the situations the model was meant to handle (such as dead end

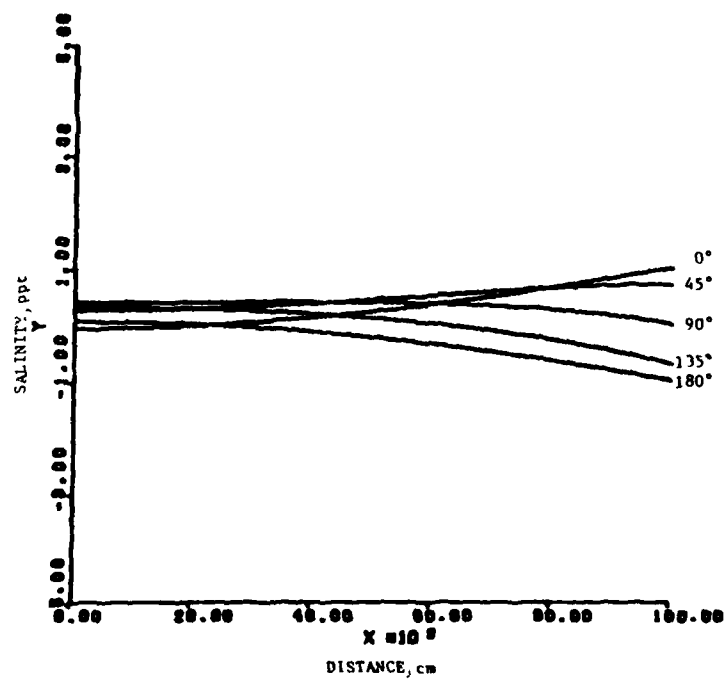
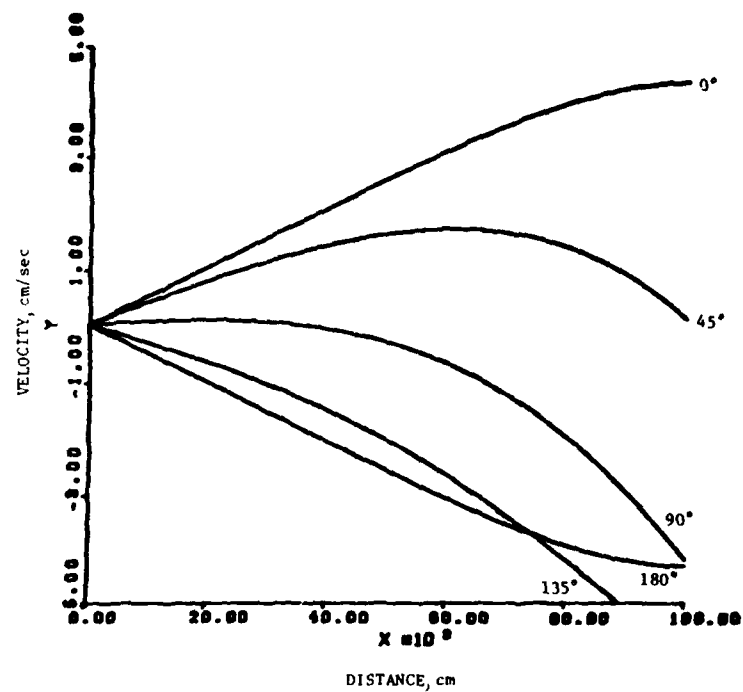


Figure IV.8. Results of Analytical Model of Density-Induced Circulation (from Monopolis, 1978)

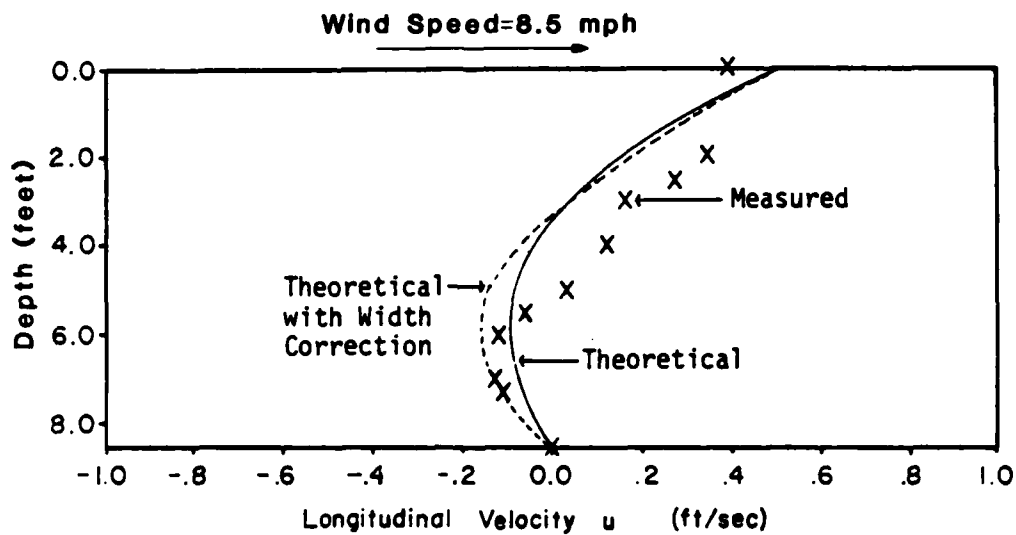


Figure IV.9. Comparison Between Model and Observed Wind- and Tide-Driven Flow

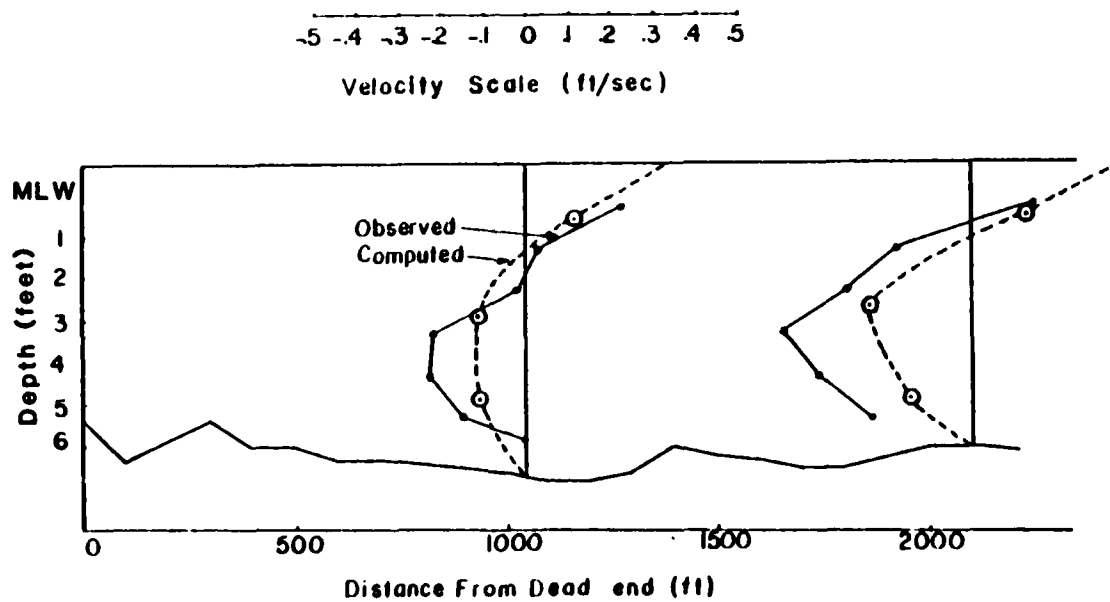


Figure IV.10. Comparison Between Model and Observed Wind-, Tide-, and Density-Driven Flow

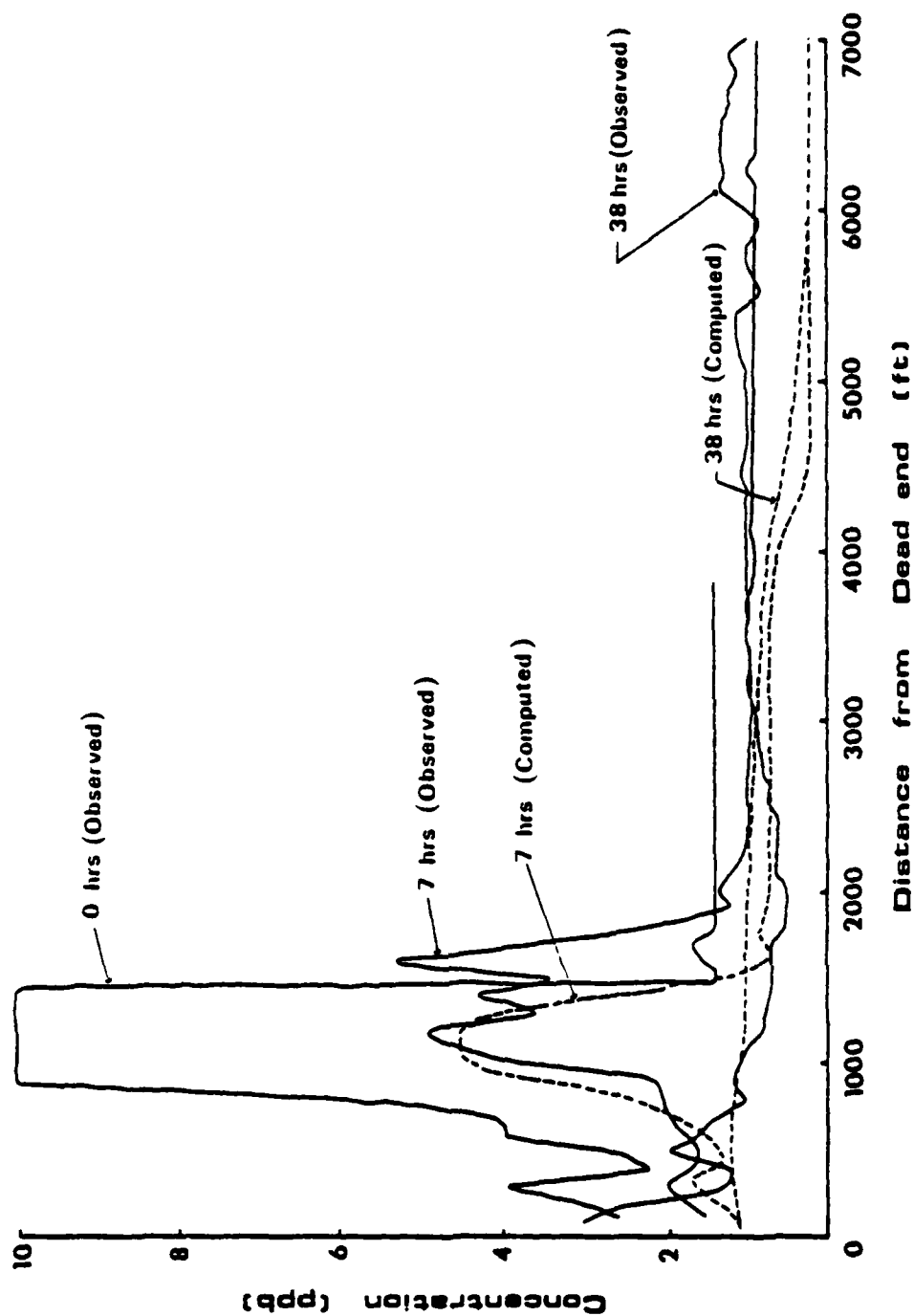


Figure IV.11. Results of Dye Simulation in 57 Acres Canal System, Florida

channels) should not be built. Another deficiency is the treatment of density currents, which was empirically based on observations. The model was, however, a useful beginning and could serve as a basis for further development.

As in canal systems, multidimensional models of marinas are rare. Most studies have used either one-dimensional approaches, or pseudo-one-dimensional, link-node models. Here again, most models are simplified, or neglect some processes over others.

A number of people have modeled the problem of shear-induced circulation in a marina basin adjacent to a large water body. Abbott (1977) numerically simulated the combination of a shear flow and an oscillatory tidal flow (Figure IV.12) and found that, to calibrate the model, he had to increase the momentum transfer coefficients in the vicinity of the basin entrance.

Askren (1979) solved the problem of a steady-state shear flow, and modeled sediment transport and deposition within a rectangular marina basin (Figure IV.13). No mass transfer was allowed at the basin entrance, which would have required the inclusion and calibration of such parameters.

Chiang and Lee (1982), Falconer (1980), and others have solved the much larger (in scale) problem of circulation within a harbor (Figures IV.14 and IV.15), but when the problem is reduced to the scale of a marina, a major drawback occurs. One could attempt to solve the governing equation implicitly. However, for Courant numbers above five, Vreugdenhil and Voogt (1975) have noted that phase errors can begin to show up in the velocity field depending upon the number of grid points/wavelength employed. For implicit calculations below a Courant number of five, it is difficult to justify the increased computation complexity involved in solving the approximate equations. Further, for an explicit solution, a Courant-governed hydrodynamic solution might require a time step, Δt , on the order of only several seconds. This is not an attractive proposition for modeling flushing rates and water quality events for an extended period of time.

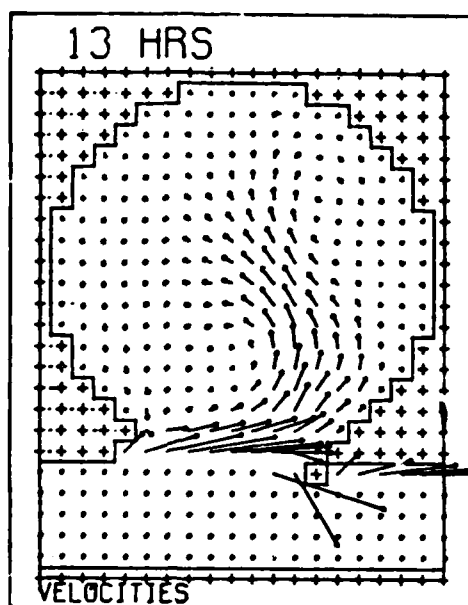


Figure IV.12. Numerical Simulation of Shear-Induced Flow in a Basin (from Abbott, 1977, courtesy of Danish Hydraulic Institute/ Oregon State University)

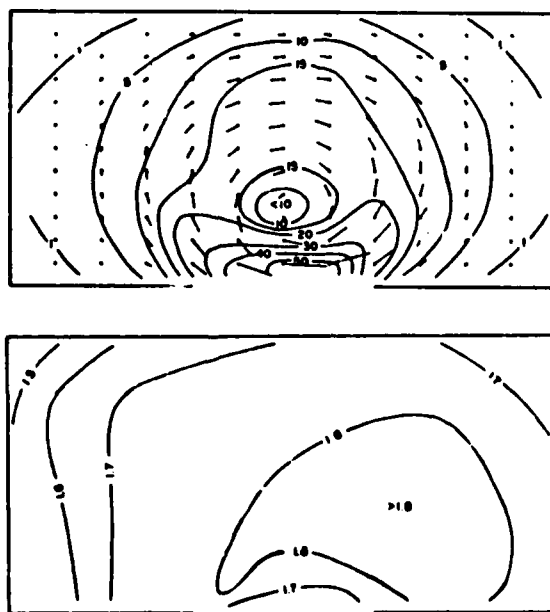


Figure IV.13. Numerical Simulation of Shear-Induced Sediment Transport in a Basin (from Askren, 1979)

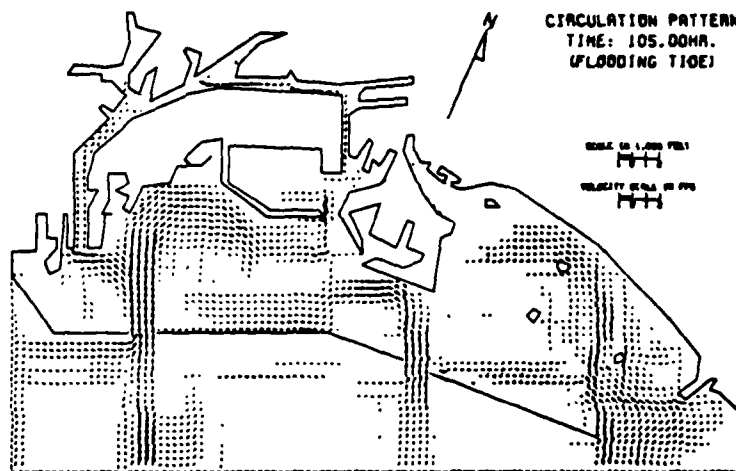


Figure IV.14. Simulated Circulation Pattern in Long Beach Harbor (from Chiang and Lee, 1982)

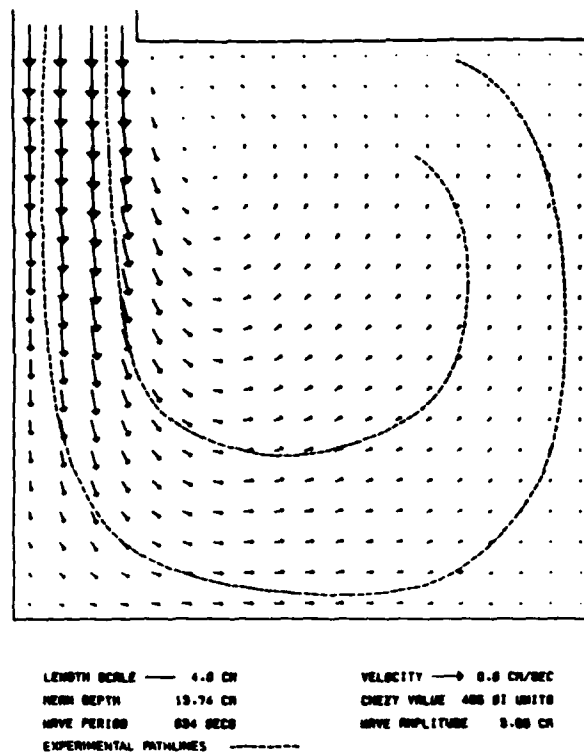


Figure IV.15. Simulated Circulation in Model Harbor (from Falconer, 1980)

The models described in this section serve to demonstrate that simple multidimensional models can be developed to simulate non-unidirectional flows. In most cases, such models have been simplified, both in terms of dimensionality and process assumptions, through judgment of observed field data. It would be useful to develop these models by the systematic reduction of a more complex model to quantify the effects of assumptions made.

IV.4 Other Approaches

Up to fairly recently, physical modeling of marinas, using tidal forcing, has generally been chosen over numerical modeling. Even considering the limitations of using distorted models for determining diffusion and mixing in tidal flows (Fischer and Holley, 1971), they probably better represent the physics than numerical models at present. Nece et al. (1972, 1974, 1975) at the University of Washington have used physical models successfully for some time. It is also interesting to note that they and Callaway (1981) state that flushing time produced from these models is in fairly good agreement with conventional tidal prism analyses.

However, Slotta (1982) points out that even such models can be very sensitive to induced circulations. As an example, he notes that the flushing time of a model marina was reduced from ten to three tidal cycles by the induced circulation introduced when dye was added. It might, therefore, be useful to study what effects wind- and density-induced circulations might have on the flushing times of marinas. These effects may perhaps be more easily studied by using a numerical model.

A completely different approach for studying canal systems was proposed by Snyder (1980) and Snyder et al. (1981). Snyder investigated the analogy between the components of electricity and oscillatory flows in canal systems. The purpose of the study was to improve mixing and circulation processes in canal systems by investigating flow-through systems or systems

that might resonate with an adjacent water body if correctly tuned. When applied to some existing systems, the method appears to work well. New systems have yet to be built to test the theory further. This approach may make an interesting tool for first-cut analysis of a proposed system, but it appears to have some shortcomings in studying effects that may cause vertical flow variability.

V. FUTURE DIRECTION

V.1 Scope

In the previous chapters, the present state of canal and marina numerical modeling has been discussed. The main conclusions from the analysis, to this point, are that we still know relatively little about such systems, and that our numerical tools are not always adequate to make constructive evaluations of proposed designs.

The understanding of the canal and marina design process needs to be improved in three areas:

1. Processes,
2. Modeling, and
3. Design criteria.

The following is a discussion of the recommended direction that future work should take.

V.2 Process Identification

Previous studies of both hydrodynamic and water quality processes in canals and marinas have been fairly limited. The nature of the data collected has been "spotty," with little effort to monitor a system over a long period of time. This effort needs to be consolidated, and a highly concentrated measurement program needs to be developed for three or four canals, and three or four marina systems, that cover the spectrum of expected situations. For residential, tidal canals, the following areas are good candidates:

1. East Coast of Florida,
2. West Coast of Florida,
3. Gulf Coast of Mississippi, Louisiana, or Texas, and
4. Coast of California.

For marinas, the following areas should be considered:

1. Puget Sound,
2. Oregon or Washington Coast,
3. Coast of California,
4. Lower Atlantic or Gulf Coasts, and
5. Upper Atlantic Coast.

In evaluating suitable choices for further detailed studies, the range of sites should exhibit the various processes we wish to quantify:

1. Tidal exchange,
2. Wind-driven flows, both in flow-through and dead end systems,
3. Density-driven circulation,
4. Geometric effects, such as multiple entrances and upstream storage areas,
5. Internal energy distributions for mixing processes,
6. Sedimentation, and
7. Water quality processes.

Each of the selected sites should be continuously monitored over the long term (perhaps one year or more) to observe the following parameters:

1. Tidal elevations at several points in the system,
2. Wind speed and direction,
3. Current speed and direction,
4. Sedimentation rates and distributions,
5. Freshwater inflow quantity and quality,

6. Water quality, as salinity, DO, temperature, etc., and
7. Atmospheric parameters, such as radiation and dew point temperature to determine effects and rates of water quality and evaporation losses.

Each system should be studied in detail by a team of scientists and engineers, and their results presented both in report form and through open discussions at a conference. The aim of this reporting procedure would be to identify the important processes of canals and marinas, and to propose methods of analytical and numerical simulations.

V.3 Numerical Modeling Approaches

Several levels of effort are recommended for numerical approaches to simulate canal and marina hydrodynamics and transport processes. The first is the development of tools that will simulate all the observed processes. This may consist of modeling hydrodynamics, sedimentation, and water quality separately, but all models should be compatible. The second is the refinement and simplification of these tools for practicing design engineers.

The development of the "scientific" models of the first group must include:

1. All processes observed from field measurements,
2. Highly accurate hydrodynamic and mass transport numerical schemes, and
3. Selection of implicit or explicit approaches.

As we have seen, many of the more common models used for canal and marina simulations neither represent the important physics nor accurately model those processes included. The aim of developing this set of highly accurate

"scientific" models is to be able to perform sensitivity analyses to determine the relative importance of terms and processes, and to see how well these models simulate prototype conditions. By starting with all the processes observed, the less important terms and processes may be eliminated and the model's ability to simulate prototype conditions noted. The ultimate aim of this approach, and a study of design requirements, would be the development of an accurate, yet conceptually simple, suite of models for canals and marinas that can be used by practicing design engineers.

It was noted in the previous chapter that process identification alone would not necessarily constitute a good numerical model. The accuracy of the numerical approximation can sometimes dominate the reliability or usefulness of the model developed. This is particularly true of the mass transport equation in low energy systems such as tidal canal networks, in which numerical dispersion can dominate that being modeled. Lee (1977), Walton (1978), and Walton and Christensen (1980) tried to overcome this problem using highly accurate numerical schemes. Unfortunately, their work was limited, and further studies are needed.

There are many estuarine models that could form the basis for the model development here. A thorough review was given by Hinwood and Wallis (1975a, 1975b). More recently, there has been quite a lot of attention paid to accurately modeling the transport equation, particularly in two-dimensional estuarine type flow. Such work (e.g. Fischer, 1978; Hall and Chapman, 1982; Holly and Preissman, 1977; Leonard, 1979; and Glass and Rodi, 1981) could form the basis of mass transport model development for canals and marinas.

In developing numerical models, some consideration should be given to investigating explicit and implicit approaches. As a general rule, explicit approaches are simpler to conceive and program. However, for canals and marinas, this approach may lead to simulation requiring very small time steps and large amounts of computer time. Traditionally, this has been overcome by using an implicit formulation. The problem here is that this

approach tends to produce significant phase errors when Courant numbers larger than five are used (Vreugdenhil and Voogt, 1975). With this limitation, there tends to be a balance between the Courant condition of explicit techniques and the extra computations involved in solving the implicit model. Some studies for reducing the phase error at high Courant numbers may be worthwhile.

Physical models have been used for many years to evaluate marina designs (Nece and Richey, 1972; Nece and Knoll, 1974; Nece et al., 1975, 1979), but none are known for canals. It would be very useful in selecting sites for comprehensive measurement programs in marinas to consider at least one for which a physical model already exists. The aim would be to compare physical and hydraulic models against field data to assess their relative merits, and to determine whether either method is adequate to describe the processes observed.

In both the canal and marina studies, it would further be useful to select sites that have previously been numerically or analytically modeled during either the design or post-design stages. Such studies would be performed to look at the adequacy of present design methods, using the more sophisticated techniques.

The design engineers require a set of models that enables them to evaluate proposed designs. They are not interested in the detailed adequacy of a particular technique or model to simulate their system, but rather an inexpensive, accurate, and reliable program to answer their questions. Design models need not contain the detailed physics of "small effect" processes, but rather are meant to look at the effect of dominant or design processes in their systems. Reduction in dimensions may also be appropriate. For example, there is a basic need to determine how well the simple tidal exchange models determine flushing rates in canals and marinas: are they useful design tools?

In developing his canal model, which was intended for use by design engineers, Walton (1978) and Walton and Christensen (1980) assumed a horizontal water surface to allow the velocity field and dispersion coefficients to be given as closed-form functions of system geometry. He also assumed a superimposition of velocities representing tidal-, wind-, and density-driven, and secondary circulations. This approach appeared to be reasonable, based on a limited number of current measurements. It would, however, be desirable to see how these types of assumptions hold for long-term observations, and to use the observations to develop closed-form functions for the various types of forcing.

The design process may call for the omission of one or more processes to evaluate the system response to "worst" conditions. As an example, van de Kreeke* considers that density-driven circulation dominates in the long term during summer months. In this situation, wind effects may be omitted, as they are generally light and variable during this period. Thus, under some design conditions, some processes may be omitted and models simplified for design evaluation purposes.

Present water quality models are generally considered over-complicated for design use in canals and marinas. Large-scale water quality models are appropriate for the group of models classified as "scientific." The design engineer, however, usually would like something less sophisticated, but adequate to address his evaluation needs. Also, as a general rule, the more complicated a model is and the more calibration coefficients are included, the harder it is to obtain a satisfactory fit between numerical results and observations. More often than not, this degree of sophistication is unwarranted in design tools, and it is recommended that some attention be given to determining which water quality processes are important for design evaluation.

* Personal Communication, J. van de Kreeke, Rosenstiel School of Marine and Atmospheric Science, University of Miami, Coral Gables, FL, 1982.

The simplest form of water quality model is the solute transport model, in which a constituent transport is included in a hydrodynamic model for the purpose of looking at flushing rates. As an example, the hydrodynamic portion of DEM (DYNHYD) is being extended (Walton et al., 1982) to look at salt and dye transport in estuaries. Such simple models are often useful for performing a first-cut analysis on circulation and flushing rates in a proposed system.

A final note is the consideration of using micro-computers as an instrument for design models. As these small machines become faster and have more storage capacity, they may be appropriate for small design models. Such a development would be very useful for the engineer in a small design firm with limited computer facilities.

V.4 Establishing Design Criteria

A useful product of numerical model development, and a distinct advantage over either the prototype or physical models, is the ability to easily perform sensitivity analyses to establish a set of design criteria. These would be useful for both the design engineer and the permitting agency. Ranges of these studies would be determined from analyses of present and recommended measurements in canal and marina systems. Limits of acceptability, in terms of poor water quality, flushing rates, and sedimentation, must be known a priori before quantifying the limit of geometric parameters to comply with these standards.

At this time, many of these standards already exist. For example, Federal regulations define what constitutes acceptable water quality in these systems. Present standards should be reviewed and revised in view of new comprehensive measurements, and should take into consideration local factors.

Once design standards have been established, a comprehensive sensitivity analysis needs to be performed to identify both the geometric limits of a given system, and also which design elements are useful in achieving the goal of environmental compatibility. Both the design engineering and permitting agency need to know:

1. How long, wide, and deep can a system be?
2. Under what conditions are side slopes stable?
3. What types of inputs can be permitted?
4. Should channels be aligned with the wind?
5. Should multiple entrances be used?
6. Should large upland water bodies, such as lakes, be connected?
7. What is the effect of connecting freshwater and saltwater bodies?

The results of studies such as these, which have been performed on some types of canal systems by Morris et al. (1978a, 1978b), and Morris (1979), will lead to a better understanding of what elements constitute a good, environmentally acceptable design. Perhaps this work can even be extended in the manner of Hayes (1982), to develop a handbook of design curves that would allow canal design to be performed without having to resort to the computer.

The computer is a useful tool for the scientist, design engineer, and permitting agency because it can be configured to run a large number of test cases with relative ease and economy. We must be careful, however, and remember that the information output is only as good as that input. Numerical models can be a tremendous asset if they are well formulated, highly accurate, and carefully proved.

VI. SUMMARY AND CONCLUSIONS

In this report, a review of modeling hydrodynamics and solute transport processes in tidal canals and marinas is presented. By considering the three areas of physical processes, modeling, and design criteria, we see that much still needs to be learned to enable the engineer to design such systems to meet a rational set of design criteria. In evaluating such proposed designs, the permitting agencies are in the same relative state of uncertainty as to what constitutes a "good, environmentally acceptable" design. Much work remains to be done to develop the science.

Future direction should include the selection of several varied existing canal and marina sites, and a program to study them to understand the fundamental physics and water quality processes involved. The next step should be to use this information to develop numerical models that can model the important processes observed, and a thorough validation of their formulation. These models should be developed as engineering, as well as scientific, tools to allow easy use by practicing design engineers. Finally, a program of selective model and measurement sensitivity should be performed to expand our knowledge of rational design criteria. These should be criteria that make sense to both the design engineer and the permitting agencies, and that lead to a framework within which both can work.

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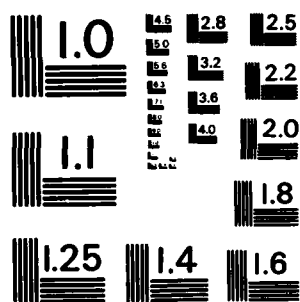
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