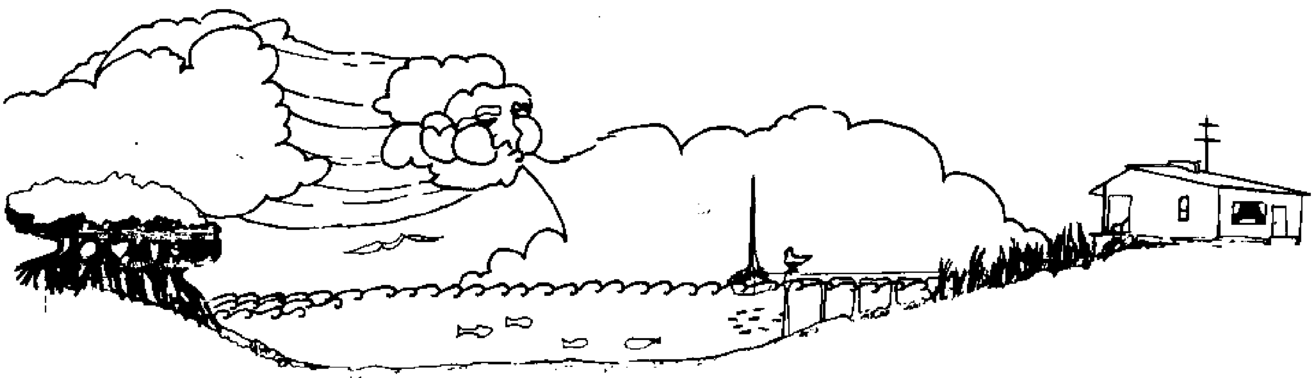
December 1981

SGR-43

Residential Canals and Canal Networks: Design and Evaluation

F.W. Morris IV



Florida Sea Grant College

RESIDENTIAL CANALS AND CANAL NETWORKS:

DESIGN AND EVALUATION

by

F. W. Morris IV

Senior Water Resource Engineer
South Florida Water Management District
West Palm Beach, Florida

Sea Grant Project No. R/OE-4
Grant No. 04-6-158-44

In cooperation with the

Board of Regents of the State of Florida
University System

and

Board of Commissioners of Palm Beach County

Report Number 43
Florida Sea Grant College
December 1981

ACKNOWLEDGEMENTS

The Northeast Regional Data Center (NERDC) computer was used for the development of the model and all of the canal design simulations. The authors particularly appreciate and wish to acknowledge the assistance provided by the NERDC operators. In addition, the Center for Instructional and Research Computing Activities (CIRCA) at the University of Florida has provided free consultation which has been invaluable.

The work described was funded by the Office of Sea Grant, National Oceanic and Atmospheric Administration, through the State University System of Florida Sea Grant College Program, the Board of Regents of the State University System, and the Board of Commissioners of Palm Beach County.

TABLE OF CONTENT

CHAPTER		Page

CHAPTER Page

LIST OF FIGURES

Figure		Page

Figure

LIST OF TABLES

NOTATION

a - tidal amplitude, (L)

A - cross-sectional area, (L^2)

A_h - Shield's entrainment function, (dimensionless)

A_{ws} - area of water surface upstream of section, (L^2)

b - bottom width, (L)

c - turbulent time mean concentration, (dimensionless)

c_I - concentration of lateral inflow, (dimensionless)

c_n - concentration at section n, (dimensionless)

c_p - initial concentration of pollutant, (dimensionless)

C - Chezy coefficient, ($L^{1/2}/T$)

c_{50} - spatial mean concentration after 50 tidal cycles, (dimensionless)

d - depth, (L)

- diameter of grain, (L)

d_e - effective grain size of bed material, (L)

d_ℓ - depth of lower interface of layer, (L)

d_o - mean tidal depth, (L)

d_u - depth of upper interface of layer, (L)

$d_{35\%}$ - the effective grain size associated with the fraction y = 0.35 finer than d

e - exponential constant = 2.718

E - longitudinal dispersion coefficient, (L^2/T)

E_p - tidal potential energy, (FL)

E_x - longitudinal diffusion coefficient, (L^2/T)

E_x' - intermediate form of longitudinal diffusion coefficient, (L^2/T)

E_y - lateral diffusion coefficient, (L^2/T)

E_z - vertical diffusion coefficient, (L^2/T)

E_o - background dispersion coefficient, (L^2/T)

f' - friction factor based on hydraulic radius, (dimensionless)

F - sum of all forces acting on a fluid volume, $(MLT^{-2}$ or $F)$

g - acceleration due to gravity, (L/T^2)

h - height of water surface above datum, (L)

H - total energy per unit weight at a specified flow section, (L)

k - Nikuradse's equivalent sand roughness, (L)

K - dimensionless dispersion coefficient (Chapter 3)

- decay coefficient, $(1/T)$

K_o - constant associated with an initial value, (dimensionless)

K_w - wind drag coefficient, (dimensionless)

K_x - longitudinal diffusion coefficient, (dimensionless)

K_y - lateral diffusion coefficient, (dimensionless)

K_z - vertical diffusion coefficient, (dimensionless)

M - coefficient for power form of uniform flow equation for mean velocity in the rough, turbulent (M) range, $(L^{1/3}/T)$

n - Manning's coefficient, (dimensionless)

- index number

N - number of tidal cycles

N_z - vertical momentum transfer coefficient, (L^2/T)

$\bar{N}_z$ - constant defined by Equation (4.33)

$N_{10\%}$ - number of tidal cycles to reach 10% of initial concentration

p - pressure at centroid of section, (F/L^2)

- point on the vertical logarithmic velocity profile

- number of straight lines (Equation 5.2)

q_I - lateral inflow per unit length, (L^2/T)

Q - discharge, (L^3/T)

r_p - rate of production or loss of substance, (1/T)

R - hydraulic radius, (L)

Re_* - wall Reynolds number, (dimensionless)

Re' - Reynolds number based on hydraulic radius, (dimensionless)

s - inverse side slope, (dimensionless)

S_b - bed slope, (dimensionless)

t - time, (T)

t' - time since low tide, (T)

T - tidal period, (T)

U - cross-sectional mean longitudinal velocity, (L/T)

u - longitudinal velocity component, (L/T)

u_m - spatial mean of u in vertical of depth d, (L/T)

u_t - tidal velocity component, (L/T)

u_* - bed shear velocity, (L/T)

u_w - wind induced velocity component, (L/T)

u_4 - constant in the salt wedge equation

v - lateral velocity component, (L/T)

V - cross-sectional mean velocity, (L/T)

$\forall$ - volume of tidal prism upstream of section, (L^3)

w - vertical velocity component, (L/T)

w_s - wind speed, (L/T)

x - longitudinal distance from upstream section of reach, (L)

y - depth, (L)

- fraction by weight of the sediment that is finer than some diameter d

z - vertical coordinate direction

- elevation about datum, (L)

Greek Letters

α - energy coefficient, or kinetic energy correction factor, (dimensionless)

γ - unit weight of water, (F/L^3)

γ_s - unit weight of bed material in bed, (F/L^3)

∂ - partial derivative operator

Δt - time increment, (T)

Δx - longitudinal spatial increment, (L)

θ - angle between wind and positive x-direction of reach, (degrees)

ν - kinematic viscosity, (L^2/T)

ρ - density, (M/L^3)

τ - time decay coefficient at tidal entrance, (1/T)

τ_o - bed shear stress, (F/L^2)

$\bar{\tau}_{cr \cdot b}$ - critical bank shear stress, (F/L^2)

$\bar{\tau}_{cr \cdot h}$ - critical horizontal shear stress, (F/L^2)

ϕ - angle of repose of cohesionless material, (degrees)

ω - tidal frequency, (1/T)

Subscripts

LT - low tide

RE - receiving water

TE - tidal entrance

Vectors are indicated by a bar, e.g. $\bar{v}$.

CHAPTER 0
READERS' GUIDE

This report has been prepared for design engineers, developers, and regulatory agency personnel interested in either designing new canal systems or rehabilitating existing ones. It describes the rational approach to canal design, the basic theory describing canal hydrodynamics and dispersion, guidelines for canal design, site surveys, canal design elements, and a procedure for evaluating an existing or proposed canal network.

It is realized that some readers may not be interested in the theoretical part, while others may only be interested in the design and evaluation technique. This readers' guide; therefore, will show where the relevant information can be found for three different groups of readers:

GROUP 1: The reader who is interested only in familiarizing himself with the potential of the procedure for evaluating a canal design should read Chapters 3 and 12.

GROUP 2: The reader who, in addition to having an interest in the procedure, has an actual project that requires coordinated planning and design guidelines should read Chapters 1, 2, 7, 8, 9, 10, 11 and 13 in addition to Chapters 3 and 12.

GROUP 3: The reader who is also interested in the theory used to develop the canal network model should, of course, read the entire report with special emphasis on Chapters 4 through 6.

RECOMMENDED READING LIST

The following list of books, reports, and technical papers are recommended for those interested in reading more about canals and canal developments, and the relevant theory underlying tidal flow in canal networks.

BACKGROUND

Barada, W., and Partington, W. M., Jr. "Report on the Investigation of the Environmental Effects of Private Waterfront Canals," Report for State of Florida Board of Trustees of the Internal Improvement Trust Fund, Feb. 1, 1972.

Carter, L. J. The Florida Experience, Land and Water Policy in a Growth State, Published for Resources for the Future, Inc., by Johns Hopkins Univ. Press, Baltimore, MD, 1974.

Chesher, R. H. "Canal Survey. Florida Keys," Report for Society for Correlation of Progress and Environment, Marine Research Foundation, Inc., Key West, FL, March 1974.

Christensen, B. A. and Snyder, R. M. "Establishment of Residential Waterfront Property by Construction of Canal Systems in Coastal Wetlands. Problems and Solutions," Paper 1301, Coastal Zone 78 Conference, San Francisco, CA, March 1978.

Environmental Protection Agency. "Finger-Fill Canal Studies, Florida and North Carolina," Report EPA 904/9-76-017, Surveillance and Analysis Section, EPA, Athens, GA, May 1975.

Polis, D. F. "The Environmental Effects of Dredge Holes: Present State of Knowledge," Report to Water Resources Administration, Dept. of Natural Resources, State of Maryland, May 1974.

Snyder, R. M. "Residential Canals and the Environment," Snyder Oceanography Services, Jupiter, FL, Oct. 1976.

PLANNING FOR DESIGN

Bureau of Coastal Zone Planning. "The Florida Coastal Management Program. Workshop Draft," Dept. of Environmental Regulation, Tallahassee, FL, Oct., 1977.

Clark, J. R. Coastal Ecosystem Management, John Wiley and Sons, New York, NY, 1977.

Corps of Engineers. "Regulatory Program of the Corps of Engineers," Part II, July 19, 1977, Federal Register, Washington, D.C., 1977.

Corps of Engineers, Florida DER, and Florida DNR. "Joint Permit Application. Dredge and Fill Structures," Tallahassee, FL, 1977.

Environmental Protection Agency. "Navigable Waters. Discharge of Dredged or Fill Material," Part II, Sept. 5, 1975, Federal Register, Washington, D.C., 1975.

Morris, F. W. IV, Walton, R., and Christensen, B. A. "Hydrodynamic Factors Involved in Finger Canal and Borrow Lake Flushing in Florida's Coastal Zone," Sea Grant Program, Report HY-7801, Hydraulic Laboratory, Department of Civil Engineering, University of Florida, Gainesville, FL, March 1978.

N.A.H.B. Land Development Manual, National Association of Home Builders, 1625 L. St. NW, Washington, D. C. 20036, 1972.

Resource Planning Section. Handbook: Building in the Coastal Environment, Office of Planning and Research, Georgia Dept. of Natural Resources, Atlanta, GA, June 1975.

Veri, A. R., Jenna, W. W., Jr., and Bergamaschi, D. E. Environmental Quality by Design: South Florida, Univ. of Miami Press, Coral Gables, FL, 1975.

DISPERSION MODELING

Aris, R. "On the Dispersion of a Solute in a Fluid Flowing Through a Tube," Proceedings, Royal Society of London, Series A, Vol. 235, No. 1200, April 1956, pp. 67-77.

Fischer, H. B. "The Mechanics of Dispersion in Natural Streams," Journal of the Hydraulics Division, ASCE, Vol. 93, HY6, Nov. 1967, pp. 187-216.

Fischer, H. B. "The Effect of Bends on Dispersion in Streams," Water Resources Research, Vol. 5, No. 2, April 1969, pp. 496-506.

Fischer, H. B. "Longitudinal Dispersion and Turbulent Mixing in Open-Channel Flow," Annual Review of Fluid Mechanics, Vol. 5, 1973, pp. 59-78.

Holley, E. R. and Harleman, D. R. F. "Dispersion of Pollutants in Estuary Type Flows," Report 74, Hydrodynamics Laboratory, M.I.T., prepared for U.S. Dept. of HEW, Cambridge, MA, Jan. 1965.

Roache, P. J. Computational Fluid Dynamics, Hermosa Publishers, Albuquerque, NM, 1972.

Taylor, Sir G. I. "The Dispersion of Matter in Turbulent Flow Through a Pipe," Proceedings of the Royal Society of London, Series A, Vol. 223, No. 1155, May 1954, pp. 446-468.

Walton, R. "Mathematical Modeling of Pollution Transport in Floridian Canals," IAHR Symposium on Unsteady Flow in Open Channels, University of Newcastle-Upon-Tyne, in association with BHRA, April 1976.

Walton, R. "Pollution Transport in Canal Networks with Small Tidal Ranges Using a Characteristic Finite-Difference Technique," A.G.U. Annual Fall Meeting, American Geophysical Union, San Francisco, CA, Dec. 1976.

Walton, R. and Christensen, B. A. "Evaluation of Pollution Transport in a One-Dimensional Canal Network Using a Method of Characteristics," First International Conference on Mathematical Modeling, St. Louis, Aug. 1977.

Ward, G. H. and Espey, W. H. "Estuarine Modeling: An Assessment," EPA, WQO Project 16070 DZV, PB-206 807, Tracor, Inc., Austin, TE, Feb. 1971.

CHAPTER 1
INTRODUCTION

While large scale coastal land development in Florida began early in the twentieth century, the construction of the extensive waterfront developments which are now seen around the coast of Florida today did not begin until after World War II. During the 1950's and 1960's an extraordinary demand for waterfront property developed, which was almost immediately satisfied by an intense exploitation of the coastline. The sudden awakening of citizen consciousness to the environmental stress brought on by the exploitative style of development, and the resulting protective legislation stemming from the National Environmental Policy Act (NEPA, 1969) brought this era to a close. Developers are now planning more environmentally and aesthetically acceptable new communities.

Since the demand for waterfront property was high and availability limited, the coastal developers planned these communities around extensive networks of branching canals. But river and estuary shorefront was expensive, so some of these developments were extended inland over thousands of acres. While some of these inland canals are completely tidal, others are isolated by control structures or salinity dams near the coastline and are fed from upland freshwater supplies. Without the tides to provide some energy for flushing, these inland canals must rely on a steady supply of freshwater for maintenance of water quality.

Some of the locations of the larger canal developments are shown in Figure 1.1. Many smaller canal communities are found along the east and west coasts of Florida, except in the wetlands north of Tampa. Since the coast of Florida has a wide variety of topographic and geologic features, as well as many different kinds of coastal ecological systems, a number of different types of residential canal networks have been developed.

1.1 Principal Types of Canals and Canal Developments

Three principle types of waterfront canal developments may be distinguished:

- *Bay-fill* or *finger-fill* canals: those constructed below mean low tide by dredging and filling shallow bay bottoms (Figure 1.2).

- *Intertidal* developments: those constructed by dredge-and-fill between mean low and mean high water; in many cases, these canals are located in mangrove or salt marsh ecosystems, in bays, estuaries, lakes, or other wetlands (Figure 1.3).

Figure 1.1 - Locations of Some Large Canal Developments in Florida.

Figure 1.2
Example of
Bay-fill Canals
in Florida.

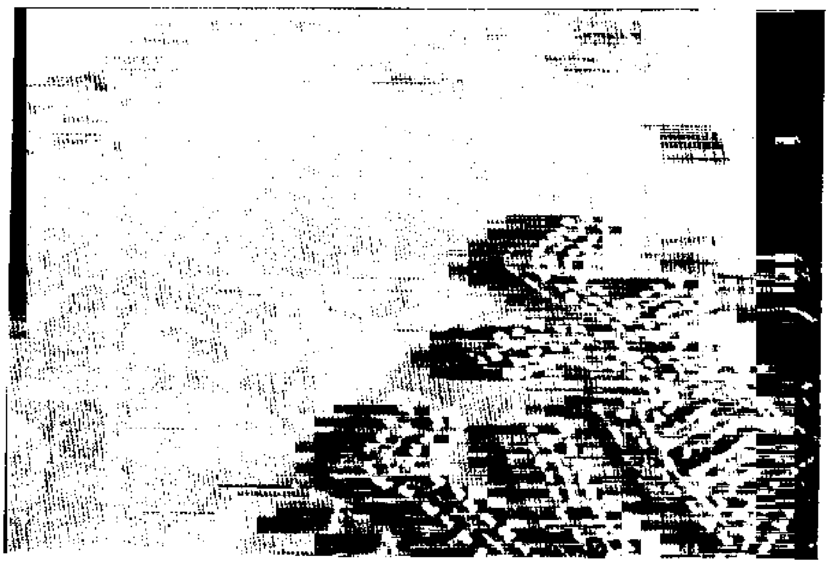


Figure 1.3
Example of
Intertidal
Development
in Florida.

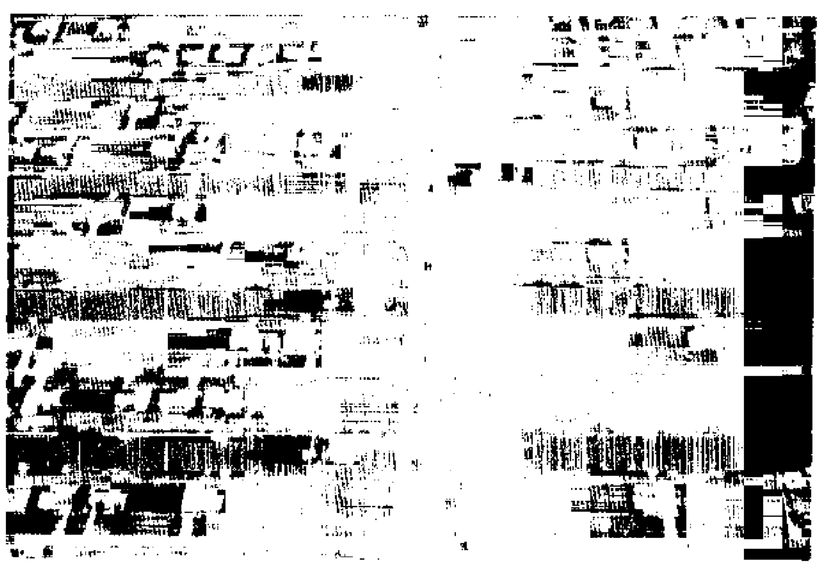


Figure 1.4
Example of
Inland or
Upland Canal
Development
in Florida.

- *Inland* or *upland* canals: those developed by excavating land which is above mean high tide and connecting the canals to natural channels, lakes, rivers, or other natural or artificial waterways (Figure 1.4).

In the past, residential canal systems were usually constructed by dredging in a manner that made the layout of housing lots and access roads most convenient, the fill being used to elevate the land surface to meet state criteria for hurricane tide and flood protection. In the process of construction, dredges excavated mangrove, grasses and trees from the channel locations and covered vegetation in the areas designated for landfill, often destroying estuarine nurseries over vast areas. In the Florida Keys, the process has been similar, although in that region the higher elevation, upland Miami oolite and Key Largo limestone substrates would first be cut with narrow, parallel, vertical ditches to a depth of perhaps ten to fifteen feet. Then the area between the ditches would be blasted and dredged into long, straight, vertical-walled channels. In the process of dredging, the bottoms of the channels were overturned and clouds of silt were carried out to nearby tidal waters, where they were deposited to smother large areas of bottom life. As a result, dredge and fill activity is now carefully regulated in Florida, and spoil banks must be located where they cannot leach into tidal waters.

A bay-fill canal network can be laid-out in any shape the developer chooses, since wetlands are flat and can be dredged at any location. Intertidal and upland canals, on the other hand, will usually be laid-out with some conformance to local topography, unless the developer is willing to pay for additional earth moving. Present federal and state regulations regarding work and construction in tidal areas require that alterations to the environment be minimized, so that designs which do take advantage of the existing topography will encounter the least resistance, both physical and political, to development.

Christensen [Christensen and Snyder, 1978] has classified existing straight canals into six major groups, and added borrow pits as a seventh category because they occur frequently in conjunction with canal development:

Group No.	Description
1	Simple dead-end canal
2	Flow-through canal
3	Comb-structured canal system
4	Higher-order finger canal
5	Canal with lagoon (basin)
6	Lagoon with two tidal entrances
7	Borrow lake

These seven classifications are diagrammed in Figure 1.5. Complex canal networks may be obtained by combining one or more of these groups, with or without curves.

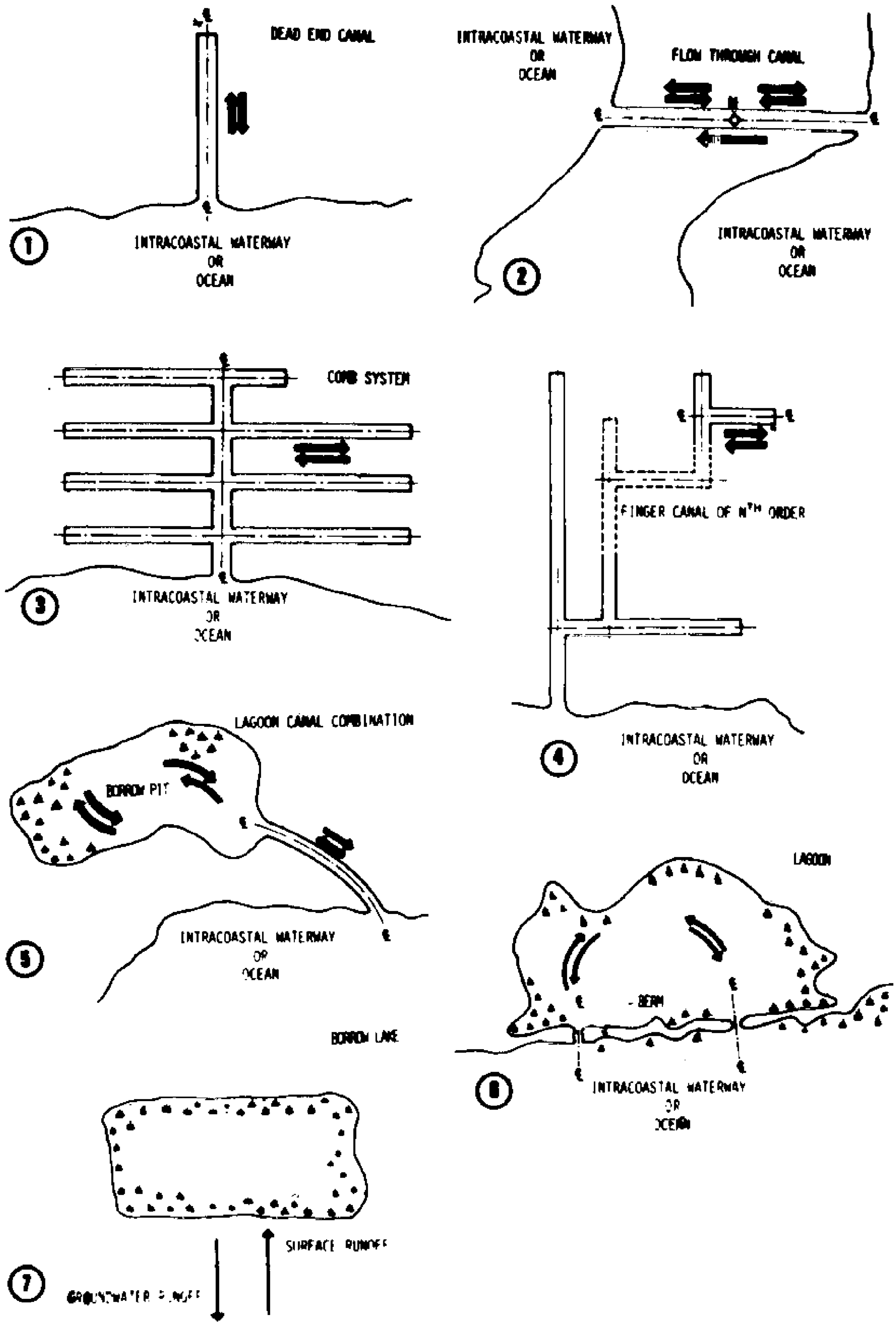


Figure 1.5 - A Geometric Classification for Types of Canals.

A simple dead-end canal (1) has only one boundary open to flow, whereas a flow-through canal (2) maintains a flow at its two open boundaries. When both entrances are tidal, and the tides are out-of-phase, the flow through such a canal provides excellent flushing. A canal network with many relatively short, parallel, closely spaced dead-end branches or fingers is a comb-structured canal system (3). These fingers may be straight or curved. A higher order finger canal network (4) is one which has one or more branches joining the main channel. If these branches are dead-end canals, the system is said to be second order. As additional branches are added to the first-level branches the order of the system increases, and it is called an upward-branching system. A canal with a lagoon or basin at one end (5), such as a marina, has somewhat special characteristics. Since the volume of water associated with the tidal prism which will flow into a tidal canal basin is a linear function of the surface area of the basin, velocities in those channels connecting a basin to the receiving waterbody will increase as the basin area is increased, which in turn will increase dispersion and flushing in these channels. A lagoon with two tidal entrances (6) has the advantages of both flow-through canals and the additional tidal prism. A borrow lake (7) has very limited circulation and depends on infiltration and runoff for purification of its waters.

1.2 Historical Canal Problems

Two physical features which have been singled-out for particular attention in many canal investigations are the depth of canal and the possible presence of a sill. When a canal is first dredged before connection to the "receiving" waterbody and a temporary "plug" is left in place,a sill is often left behind when the plug is removed. Generally, however, the term sill may apply to any relatively shallow section at any location in the canal which impedes the circulation in the bottom waters inside the canal.

It has been observed that "deep" canals are not adequately flushed by tidal action and that the lower layers act as a trap for sediments and organic detritus. Polis [1974, p. 23] and Barada and Partington [1972, p. 10] reported results of an investigation in which thermal stratification was formed in all canals investigated which were deeper than fifteen feet. A sharp density interface was measured at depths between ten to twelve feet in such canals, with indications of less turbidity, anaerobic conditions, and the presence of hydrogen sulfide in the region below the interface. It has also been observed that canals that are too shallow may also have poorer flushing characteristics, and have limited navigability as well.

In searching for a simple method by which "good" and "bad" canals can be separated, governmental agencies have found that the vertical dissolved oxygen profile (or surface and bottom values) can be related statistically to the mean depth. Thus, the Environmental Protection Agency (EPA) 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 (DO) profiles and numerical flushing models [EPA, 1975a, p. 5]. However, the one-dimensional dispersion model used in making this determination, which is based on very restrictive assumptions, and on only the cases that were simulated, was not realistic enough to encourage wide acceptance of this oversimplified criterion.

A sill at the entrance to a canal acts as a trap for the bottom, denser water and fluidized sediments, and suppresses vertical mixing for some distance into the canal. Since vertical mixing is the principal means by which reaeration of the bottom waters is effected, the accretion of flocculent sediments and organic detritis results in a sustained demand for oxygen which can lead to anaerobic conditions and the release of hydrogen sulfide. The same effect, on a smaller scale, occurs in deep holes in the beds of canals, which can be caused by improper dredging for fill. Effective wind mixing can reduce this problem somewhat, but it is generally recommended that sills be removed from such canals.

Improper canal construction can also significantly affect coastal aquifers and drainage. The aquifers, or underground freshwater storage areas, are characterized by an interface with the seawater that intrudes into the aquifer a distance underground which depends on the potentiometric head above the interface (Figure 1.6). As this head decreases, the saltwater interface moves upward and inward farther into the aquifer. This relationship is known as the Ghyben-Herzberg principle, which demonstrates that the depth of the interface below mean sea level is about thirty-eight times the height of the freshwater table above it. Thus, a canal may bring tidal waters farther inland (Figure 1.7), and sometimes may substantially increase drainage from inland areas, both of which can significantly increase saltwater intrusion. This effect, however, may not be observed until many years after the canal system is opened to the tide since the flow through the porous aquifer is extremely slow.

The quality of the water in tidal canals can be characterized by many different chemical and/or biological parameters. Whether it is described by measurement of dissolved oxygen, or whether it has been indirectly indicated by a fish kill, the water quality has been observed to be degraded in many canals in nearly all regions of the state. This occurs primarily when the water circulation, and the resulting flushing action, are not of sufficient magnitude throughout the canal network to maintain dissolved oxygen throughout the water column and to carry undesirable pollutants out to the receiving waterbody. Thus, since water circulation is the basic mechanism for maintenance of water quality, canal design must first concentrate on a comprehensive description of the hydrodynamics in the canal system.

In canals with oxygen-depleted bottom waters, most aquatic life can only inhabit the upper, oxygenated layer. If these bottom waters are suddenly driven to the surface, as might happen if a storm with strong winds oriented in the direction of the canal channel were to induce upwelling, mass mortality of these aquatic organisms could result.

Another problem with canals has been bank and bed instability. Velocities which are too low result in the deposition of sediments, particularly in deep holes. Sediments may consist of sand and clays eroded upstream by faster-moving currents, or organic material consisting of dead aquatic life or plant detritus.

Vegetation acts as a natural zone for deposition since velocities become very small among the roots and stems of aquatic plants. These zones are often referred to as "nutrient traps". A certain nutrient flux is required for growth of a particular type of vegetation, but excessive nutrients can

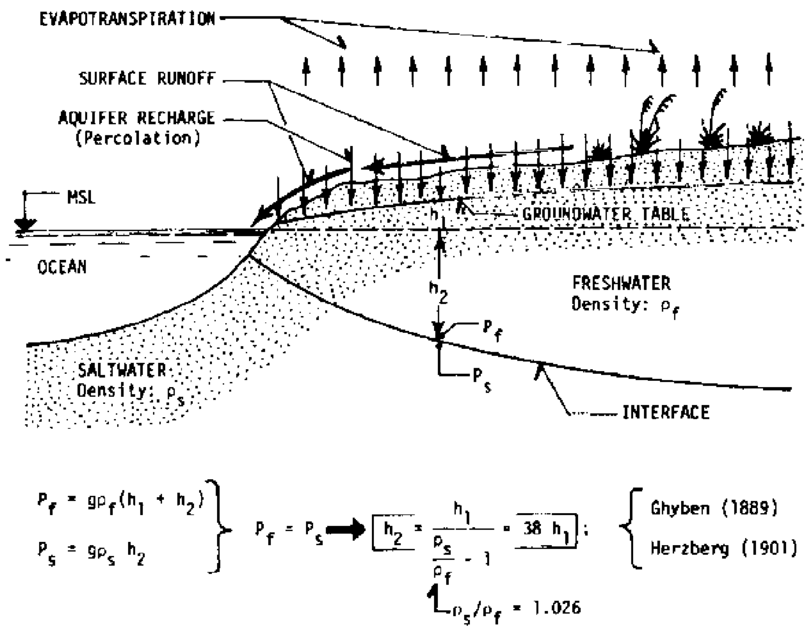


Figure 1.6 - Undisturbed Coastal Phreatic Aquifer in Equilibrium.

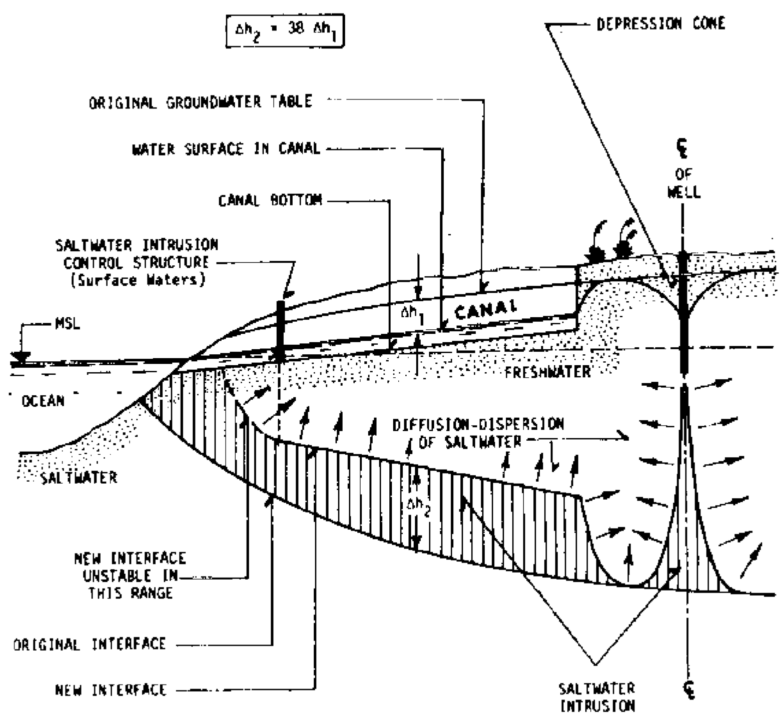


Figure 1.7 - Increased Saltwater Intrusion due to Canal and Well.

lead to algal blooms, oxygen depletion, fish kills, and subsequently worsening conditions.

Water quality in canals is also adversely affected by pollutants introduced from various sources along the boundaries of the canal system. The major sources of pollution are stormwater runoff, septic tanks, sewage treatment plant effluent and boats and houseboats. The contents of these pollutants vary widely and have been the subject of numerous studies.

Stormwater runoff contains materials which collect on streets, roofs, and lawns and are channeled in drains, storm sewers, and drainage ditches. They include chemicals such as insecticides, herbicides, and fertilizers; animal wastes and sewage; oil and grease; chemical products from cleaning operations; garbage, refuse and trash; and dead or dying vegetation washed into the canals by storms. These pollutants are either flushed out or accumulate on the surface or in bottom sediments, depending on the flushing ability of the canals.

As of 1972, "more than half of Florida's canal-type developments utilized septic tanks for municipal sewage disposal" [Barada and Partigan, 1972, p. 20]. In many systems, however, either soil conditions or the elevation of the canal waters reduce the efficiency of septic tanks to such a degree that virtually untreated sewage is being leached into the waterways through the sides of the canals. In addition, septic tanks in locations with high water tables are often prone to overflow during heavy rains.

Sewage treatment effluents from both public and private plants evidently have been a problem in many canal systems in Florida. As communities expand, these facilities often are not upgraded and easily become overloaded. While legislation provides definite guidelines and limits for the operation of such facilities, these usually stop at requiring secondary treatment. This criterion still does not remove dissolved phosphates, nitrates, and other chemical contaminants, which in many cases are oxygen-demanding materials.

Boats and houseboats have, in the past, been permitted to discharge sewage directly into the canals and have been a source of gasoline and oil wastes, bilge-water, garbage, and refuse as well. Regulations in particular areas may require pump-out facilities for household wastes, since pollution from petroleum products cannot be effectively controlled by relying only on canal flushing.

The documentation of canal problems in the 1950's and 1960's resulted in restrictive legislation, and a significant decrease in canal construction. The direct causes of degradation in water quality were, in most cases, clear, but a rational approach to the design of canal systems did not begin to emerge until studies of canal behavior and hydrodynamics were initiated in 1974. Chapter 2 explains the interrelated causes of canal problems, and presents a rational approach to comprehensive design.

CHAPTER 2
RATIONAL CANAL DESIGN[1]

The degradation of the environment caused by some canal developments in the past occurred more from a lack of understanding of the physical, chemical, and ecological principles characteristic of tidal marshlands and waterways than from intentional misuse. The goal of previous canal design was simply to provide as much waterfront property for residential units as was feasible, at the least cost. Certainly, conscious efforts were not made to destroy the environment upon which these canal developments depended. There is no doubt, however, that some developers were primarily concerned with obtaining a quick turnover to maximize their profits, with little regard for the short-term effect on the environment. Furthermore, the long-term effects of such activities were not at that time generally recognized. What design guidelines were available pertained more to the techniques of construction and the legislation governing civil works in urban areas than to works located in the fragile and vulnerable wetlands.

With the recognition of the long term negative environmental influences resulting from early canal design, the entire approach to development of the coastal zone has had to be reviewed to determine if new criteria can be developed that will permit utilization without degradation. Control of dredge and fill techniques is not sufficient to avoid adverse environmental effects. It is now recognized that, in order to predict the environmental impact of a particular project, the entire ecosystem involved must be studied. In particular, with regard to the development of residential canals, one may begin by attempting to determine the direct causes of the well-documented environmental degradations that have resulted from past construction, and then ask whether or not a different approach can be devised that could result in healthy and productive canals.

2.1 Early Canal Design, and its Contribution to Environmental Degradation

The ultimate effect of early canal design was, in most cases, the degradation of water quality and biological habitat. In the worst of these cases, the cause of water pollution was the direct influx of partially treated sewage or industrial wastes into the canals or adjacent waters. In many areas, seepage from septic tanks was carried to the canals by groundwater flow. As a result, direct discharge of sewage or industrial effluent is now prohibited by federal law. Many counties and states now also prohibit using septic tanks in waterfront developments. In addition, as a result of studies for the Envrionmental Protection Agency, (EPA) on the pollutant content of runoff over streets and roads, regulations have been developed prohibiting direct discharge of stormwater runoff into receiving waters.

[1]Most of the material in this chapter is excerpted from a report by Snyder, [1976].

Elimination of these direct pollution sources immediately changes the water quality picture in residential canals. It is even possible that residential canals designed with the early criteria, in the absence of direct pollution sources, could maintain water quality at or above the level of state standards. Recent studies of canal hydrodynamics, however, indicate that a great deal more can be done in the way of design to not only maintain water quality, but to improve the quality of the receiving waters and provide significant additional areas of biological habitat.

The ultimate effect of the early approach to canal design is diagrammed in Figure 2.1. This shows how the goal of maximizing "residential utilization of waterfront" leads to "degradation of water quality and biological habitat" when the listed incidental factors and engineering criteria are applied to a particular design. The expected environmental impact of each of these early engineering criteria may be explained as follows:

ENGINEERING CRITERIA

a) *NAVIGABLE DEPTHS TO SHORELINE* - Use of this early design criterion leads to elimination of intertidal shallows and the placement of boat berthing facilities anywhere within the canal system without regard to location of lot lines or centralized facilities. A cursory inspection of typical bulkheaded canals will show that no more than 20 to 30 percent of the bulkhead space is required or used as berthing space by the residents. Even in southern Florida, where boating is a 365-day-a-year activity, it is unusual to find even a 20 percent utilization of bulkhead space. The requirement for boat berthing, then, should not be translated into the requirement for navigable depths to the shoreline. This criterion should be eliminated as a requirement for residential waterfront utilization.

b) *MAXIMIZE FRONT FOOTAGE PER ACRE* - This criterion was obviously adopted to maximize profits, and has been shown to be inconsistent with environmentally sound development. It should no longer play a part in development planning.

c) *INCREASED ELEVATION FOR FOUNDATIONS* - Increased elevation is often required by site topography as well as by legislated building requirements in those areas in which the residential area must be raised to the elevation of the 100 year (or other) flood plain. There is nothing wrong with using material dredged from canals to provide land fill for this purpose, if the resulting canals are otherwise environmentally acceptable and the depth variations, if any, are approved by the canal designer. Fill also may be obtained from dredging shallow freshwater lakes within or adjacent to the project that would subsequently be used for stormwater retention and groundwater recharge. Additional fill, if needed, can be brought in from offsite. The canal design, however, should not be dictated by project fill requirements, since it may lead to uncontrolled irregular and excessive channel depths.

d) *MINIMUM LOSS OF PROPERTY TO WATER AREA* - This criterion was also adopted to maximize profits by maximizing the number of waterfront lots per total project acreage. It is not consistent with environmentally sound development and should be discarded as an engineering criterion.

e) *RAPID DRAINAGE OF RAINFALL* - This criterion resulted from accepted civil engineering practice dating back a number of centuries, water supply and drainage being prime requisites for urban development. Because of the pollution potential of street runoff, however, and because of potential salt-water intrusion problems, runoff should not be released to the canals but rather should be retained on site and used for groundwater recharge. On project sites located near the terminous of large drainage basins there will, of necessity, be some direct freshwater runoff into receiving waters. Because this is a natural phenomenon it is not directly injurious to the receiving waters.

f) *SIMPLIFIED SURVEYING AND CONSTRUCTION METHODS* - Simplified procedures result in channels running in straight lines, with right angles, and level topography. They reflect typical urban or suburban land platting methods carried over to residential canal design. These characteristics are often associated with low cost housing developments and are certainly not consistent either with environmentally sound development or with aesthetic land planning. Modern materials and construction techniques have made land utilization more flexible and should be fully utilized in the design of residential canals.

The above engineering design criteria were not necessarily formally adopted by developers or engineers, but resulted from the lack of knowledge of environmental impact. By analyzing the mechanisms of environmental degradation, positive design criteria that will maintain, or even enhance, environmental quality in developing residential waterfront within the coastal zone have been established. It is known that healthy systems do exist. Therefore, if cause and effect can be determined, it should be possible to design and construct healthy canal systems.

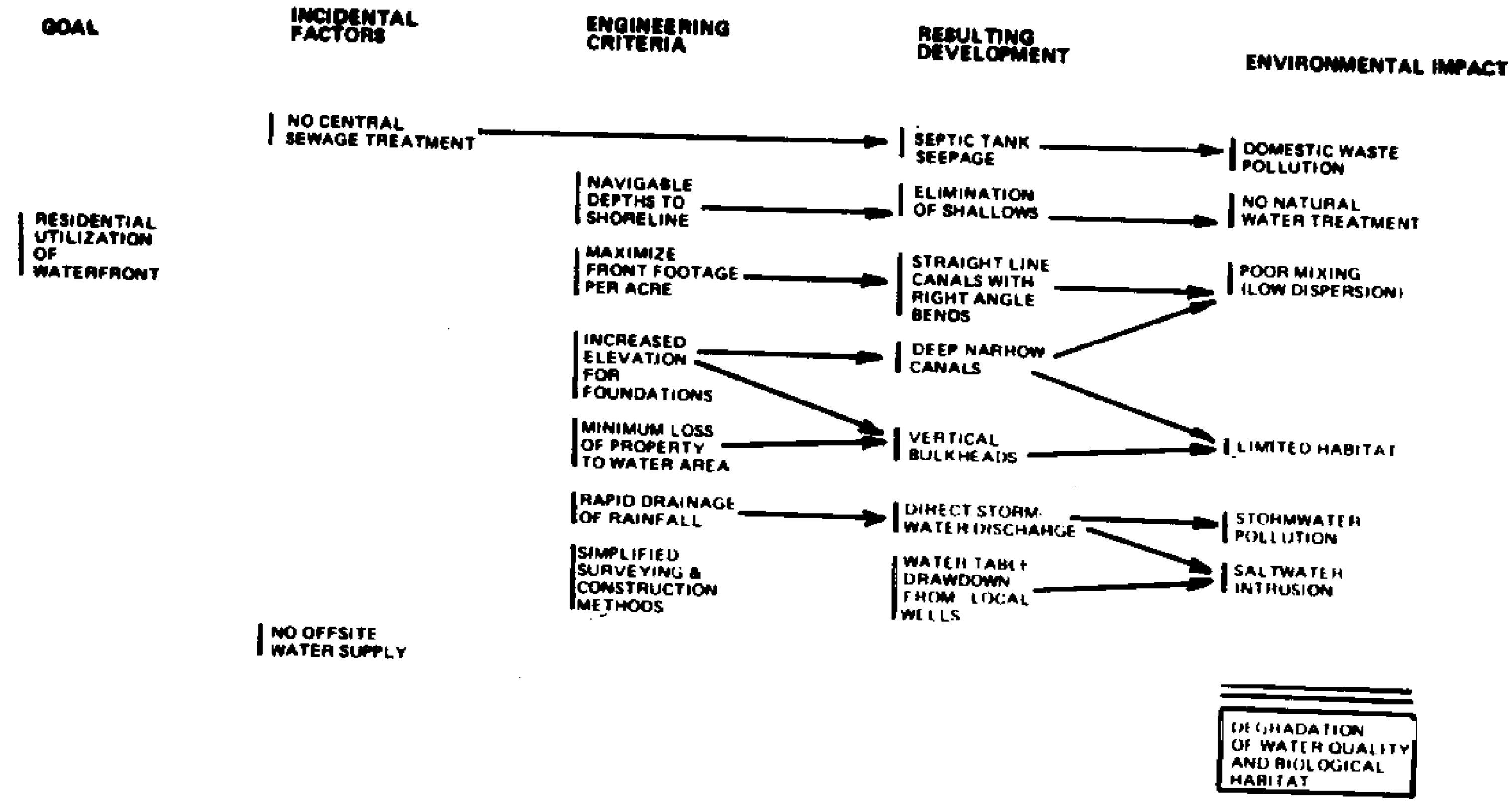
Continuing with Figure 2.1, it can be shown that the early design approach results in the following characteristics in many developements:

RESULTING DEVELOPMENT

a) *SEPTIC TANK SEEPAGE* - This is a problem if soil conditions are not correct, and/or if the septic tanks are located too close to the waterway. If local building ordinances do not prohibit septic tanks in waterfront developments, then permitting agencies usually provide rigid standards that must be met by the developer.

b) *ELIMINATION OF SHALLOWS* - Shallow areas are created at the shoreline when the sides of canals are sloped. When these slopes are vegetated, they have a self-purification capability through biological action. The elimination of shallows by bulkheading thus removes much of the capability of the canal system to purify its own waters by natural means.

c) *STRAIGHT-LINE CANALS WITH RIGHT ANGLE BENDS* - Secondary currents, which are movements of water in directions other than in the longitudinal direction of the primary flow, are substantially smaller in straight channels than in channels with bends. Such currents are the primary means for maintaining vertical mixing, which replenishes the bottom waters with oxygenated surface waters and organics for food, and brings nutrients from the bottom to the surface. Conversely, right angle bends are wasteful of the limited energy in the tidal flow, which should be distributed and used for mixing and transport of pollutants out of the canal system. Straight canals with right



SNYDER OCEANOGRAPHY SERVICES

Figure 2.1 - Early Canal Design.

angle bends, therefore, are not recommended as they do not properly utilize the available energy to promote flushing.

d) *DEEP NARROW CANALS* - Canals that are deep and narrow tend to compound the problems discussed in the previous two paragraphs, primarily because the effect of wind on vertical circulation is substantially reduced by decreasing the width of canals, as well as by trees along the banks. The potential for natural mixing is thus reduced and the water can eventually become stratified.

e) *VERTICAL BULKHEADS* - Vertical bulkheads, in themselves, are not detrimental to the environment. In fact, they offer effective attachment surfaces for sessile organisms and commensal communities that might otherwise be absent. Absence of intertidal areas which are provided by sloping banks, however, severely limits the diversity of habitat within a canal system. Many projects contain miles of vertical bulkheading with no relief and, in addition to being unnecessary for effective utilization of the waterfront, the continuous bulkheads exclude the possibility of the development of a balanced ecosystem. There is justification, then, to limit the use of bulkheads to areas of boat berthing and where structural design (e.g., bridge abutments) dictates their necessity.

f) *DIRECT STORMWATER DISCHARGE* - There are two reasons why direct discharge of stormwater into canals under normal conditions should be prevented or discouraged. The first is that initial runoff will have high concentrations of pollutants, and the second is that water so discharged is lost to the groundwater system. A development of any kind increases runoff because of the many artificial surfaces (such as rooftops and roads) which are introduced, and reduces natural recharge of the groundwater. If stormwater is collected and directed to surface or underground recharge basins, three positive environmental results are realized. First, less fresh groundwater will flow into the project site and eventually into the canals. This preserves the offsite groundwater resource. Second, the stormwater is naturally purified as it moves through the soils toward the canals. This prevents a high concentration of street contaminants from entering the receiving waters through groundwater exchange. Third, as shown by the Ghyben-Herzberg relation, for every addition of one foot of freshwater head the saltwater interface will be depressed about forty feet downward, thus inhibiting saltwater intrusion.

g) *WATER TABLE DRAWDOWN FROM LOCAL WELLS* - As above, for every foot of freshwater head removed from the local groundwater table because of draw down from local wells, the saltwater interface will rise about forty feet, leading to eventual contamination of the local water supply. Offsite water supply and stormwater recharge, then, combine to offset the saltwater intrusion that might otherwise result from canal construction.

The environmental impact of canal developments using the preceding design criteria are readily visible or measurable. They include domestic waste pollution, the lack of natural water treatment, poor mixing, limited habitat, stormwater pollution and saltwater intrusion. Because of the resulting environmental degradation, most regulating agencies will prohibit the construction of any new residential canals until it can be shown that the canal system is compatible with the site, that it does not degrade the environment, that it meets all regulatory criteria, and that it is designed rationally.

2.2 A Rational Approach to Residential Canal Design

Based upon the scientific and engineering literature, theoretical developments, laboratory experiments, field verification studies, and inductive reasoning, a new set of canal design guidelines and criteria is proposed in this manual. These guidelines and criteria have been used in a number of residential canal design projects and research is continuing to refine and quantify virtually all of the environmental and engineering factors involved, so that accurate predictions of proposed developments can be made based upon measurable physical, chemical, geological and biological parameters of the project area. This RATIONAL APPROACH is presented in Figure 2.2.

The GOAL of the rational approach is RESIDENTIAL UTILIZATION OF WATERFRONT WITHOUT ENVIRONMENTAL DEGRADATION. By recognizing the potential for negative environmental impact, and modifying the goal to include total environmental concern, a methodology can be developed for use in all areas that will allow environmental modification without degradation.

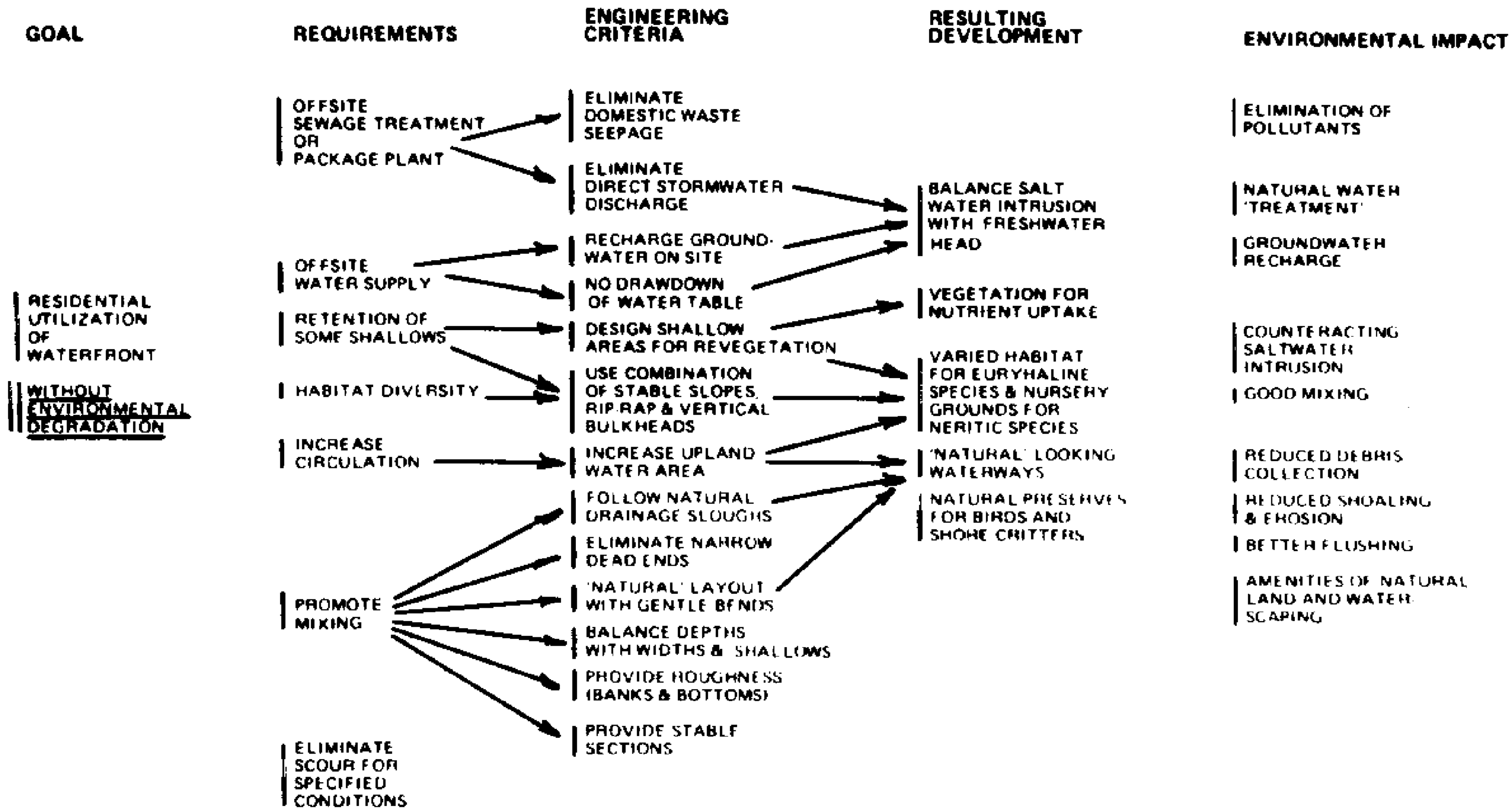
In Figure 2.2, the *INCIDENTIAL FACTORS* of Figure 2.1 have been replaced by a set of *REQUIREMENTS*, some of which have resulted from legislation, codes and guidelines adopted by local, state and federal agencies. Others, such as retention of shallows and habitat diversity, are inductive inputs resulting from the study of early canals in relation to natural systems.

REQUIREMENTS

a) *OFFSITE SEWAGE TREATMENT OR PACKAGE PLANT* - This, in combination with discharge regulations, prevents the direct pollution of canal and receiving waters by domestic sewage. Use of offsite facilities for sewage treatment moves the problem to another jurisdiction where it may, or may not, be handled properly. If an offsite facility is selected, its adequacy should be investigated. Package plants can have both positive and negative effects. On the negative side, the disposal of the effluent can still pollute the environment through malfunction, faulty operation or neglect. Thorough studies should be conducted of the proposed disposal method including groundwater flow, appropriate storm statistics and basin hydrographs. On the positive side, the increased use of package plants (mostly on an interim basis) has led to design and construction improvements that obviate problems with earlier units. A properly designed and operated package plant will enhance the local groundwater resource, helping to control saltwater intrusion.

b) *OFFSITE WATER SUPPLY* - Use of offsite domestic water does not deplete the local groundwater resource. In fact, the domestic water may be recharged through the package plant system yielding additional groundwater flow, adding to canal flushing volume and balancing saltwater intrusion.

c) *RETENTION OF SOME SHALLOWS* - Shallow, intertidal areas are important parts of any productive natural estuarine system. Intertidal vegetation consumes nutrients from the water, and the detritus from this vegetation serves as the basis of the food web in estuarine areas. There is no reason to believe that a properly designed canal cannot be as productive as an estuarine channel both in the production of food and in the "self-purification" of the canal waters. Under certain conditions the canal could be expected to improve the quality of receiving waters by reducing nutrient content and



SNYDER OCEANOGRAPHY SERVICES

Figure 2.2 - The Rational Approach to Canal Design.

possibly raising the level of dissolved oxygen. Evidence exists that much of our intertidal vegetation is nutrient starved and hence capable of reducing nutrient levels.

In addition to supporting intertidal vegetation, the shallow areas are better suited for support of benthic flora and fauna which can contribute to system productivity.

A third beneficial influence of intertidal shallows is the pumping effect of the tides which forces canal water through the earthen embankments, providing biological filtration. All these areas need further study to determine the degree of their influence on water quality and productivity. Lack of quantified data, however, in no way alters the basic physics or biochemistry of the processes involved and few would argue that the presence of intertidal shallows would not have a positive impact on water quality and productivity.

d) *HABITAT DIVERSITY* - As with the influence of intertidal shallows, the benefits of habitat diversity are recognized but are not presently quantifiable in terms of canal design parameters. Induction from observations of natural habitats, however, indicates that the relative areal extent of aquatic habitat might be placed into the following hierarchical arrangement:

- VEGETATED INTERTIDAL SHALLOWS
- SLOPING RIP-RAP
- VERTICAL BULKHEADS

The optimum relationship between these three bank configurations will depend upon the area in which the canal is to be constructed and the indigenous flora and fauna.

e) *INCREASE CIRCULATION* - Circulation, the passage of water through the system, has long been considered important to help flush pollutants from the system. With both point and non-point sources eliminated, circulation is still important for the distribution of nutrients, dissolved gases and food particles. Circulation is a function of the tidal dynamics, wind in the area, and the design of the canal system. Freshwater inflow, both through the soil and over control structures, also contributes to circulation, albeit in a more complex way. The circulation in any canal system may be increased by increasing the water area inside the canal system. The tidal prism, ie. the area of water surface times the tidal range, that is required to fill an inland basin must traverse through the interconnecting canal. The greater the inland tidal prism, the greater the circulation in the connecting canals. Canals with two or more openings into the receiving waters can, if designed properly, and if there is a tidal differential, exhibit greater circulation than dead end canals. The combination of greater inland area and multiple openings can often provide the greatest circulation.

f) *PROMOTE MIXING* - The degree of mixing is a function of circulation, but it is also heavily dependent upon physical design factors in the canal. In a straight, dead end canal, the tidal range may be fully experienced at the dead end. This does not mean, however, that new water from the receiving body will reach the dead end on each tidal cycle. Nor does it mean that the water at the dead end will be released to the receiving body during a tidal period. The exchange rate from the dead end can be extremely slow, amounting to a few percent dilution per tidal cycle. This dead end dilution rate can be increased by inducing turbulent mixing and the generation of secondary currents.

g) *ELIMINATE SCOUR FOR SPECIFIED CONDITIONS* - With the elimination of continuous bulkheads, and the inclusion of shallow, intertidal areas, bank and bed stability must be considered. Stability is a function of the canal volume, cross-sectional area, tidal prism, slope of the banks, and bank and bed material. Criteria for design should be specified by choosing storm conditions of a certain frequency along with an acceptable probability of erosion. The criteria can be specified by statute, agency guidelines or determined on the basis of engineering or other environmental considerations.

It should be obvious that canals developed under the above criteria will look and behave entirely differently from those constructed according to the early criteria, or lack of criteria. The environmental impact of canals designed following this rational approach will include elimination of pollutants, natural water treatment or self-purification, groundwater recharge, balanced saltwater intrusion, good mixing, reduction of debris collection, reduced shoaling and erosion, better flushing, varied habitat for marine species, natural preserve areas for shore and other birds and, for the residents, the amenities of natural land and waterscaping.

Designing residential canals that will not degrade the environment will very likely result in systems that are an asset to environmental productivity. This would be especially true if upland areas, requiring very little filling or modification of wetlands, are used as project sites.

CHAPTER 3
IMPLEMENTATION OF RATIONAL DESIGN

Rational design is the application of science, engineering, and common sense to the conflicting requirements for environmentally-compatible development. It requires an understanding of the principles of physics (in particular, hydrodynamics), chemistry, biology, ecology, geology and, of course, civil engineering construction. The design must be quantitative. Thus, the canal designer must not only be able to visualize and synthesize a working hydrodynamic system that will fit into the characteristics of a particular site, but he must be prepared to predict, quantitatively, the operating characteristics and effects of his design.

Building on the picture of rational versus non-rational design presented in Chapters 1 and 2, this chapter outlines the implementation of the rational approach, which is then further detailed in the ensuing chapters.

3.1 The Distribution and Interchange of Substances in Canals

A variety of substances are found in waters of all canals. Some substances are called "pollutants" because certain aquatic life cannot function normally in their presence, or because people find them offensive or hazardous to their health. Coliform bacteria, which most commonly occur in sewage, and algae, pesticides, and ions of heavy metals are examples of these kinds of pollutants. Other substances may not constitute a direct health hazard, such as floating grass clippings and trash, but may contribute to a general degradation in water quality over a period of time. Still other substances may actually be beneficial, such as clean freshwater inflow. Thus, for purposes of designing acceptable tidal canal networks, a pollutant may be considered to be any substance or constitutent which would not normally be found in the communicating waterbody and which is considered to be present in sufficient concentration to constitute a hazard to aquatic or human life.

Since there is a great variety of potential pollution sources, it is not practical to attempt to analyze or predict the influence of each possible constitutent on the waters of a given canal system. It is useful, however, to determine a representative pollutant loading at all locations in the canal system and to predict the ultimate concentration and distribution of that pollutant under the action of normal hydrodynamic and environmental variables, for purposes of comparing alternative canal network designs.

The distribution of concentration of a pollutant in a tidal canal system changes continually. It is influenced by the circulation of water, the location and rate of inflow of pollutants, and the loss of pollutants in the beds and banks of the channels and through the tidal entrance. Each of these factors, in turn, varies continually in response to the tide, climate, and activities of the residents of the community.

The effectiveness of a particular canal network in utilizing natural energy sources for the removal of pollutants is indicated by its flushing characteristics. Since the rate of removal depends on the spatial change in pollutant concentration, the water velocities, and the rates of inflow and loss of pollutants at every location in the network, the flushing characteristics also are not constant. It is possible, however, to predict the pollutant concentrations in a particular canal network over successive tidal cycles, for a well-defined set of conditions. This kind of prediction is accomplished by simulating the flushing action using a computer model. If the canal designer or evaluator can specify the variation of all of the independent variables over the time span to be considered, and selects a criteria for judging the effectiveness of flushing, than the results for different canal network geometries and pollutant loading rates over a specified number of tidal cycles may be compared for realistic tide, climate, and pollutant loading conditions. Examples of such design comparisons are presented in later chapters.

It should be remembered that a proposed canal system is only a small part of the much larger, intricately connected land, water, air and biological systems in its vicinity. Therefore, the design must take into account at least all of the principal characteristics and interrelationships of these far more extensive systems. For example, it will not usually be acceptable to simply flush all pollutants as efficiently as possible from the canal network to the communicating tidal waterbody. The impact of the canal network on neighboring land, water, and biological communities, as well as those on the site itself, will be scrutinized at all levels of review of the project.

3.2 Canal Design

A canal design is a set of specifications describing the desired layout, appearance, and operating characteristics of a particular residential canal development plan, which covers such details as the layout of the canal network, the geometry of the canals, the predicted flushing time of the network, and the design of tidal entrance(s).

While the design of the canal network is a relatively small part of the design of a residential development, it is a key element in such a plan because the canals must be designed to flush efficiently and solely in response to the natural conditions present at the site. The canal network design must therefore be integrated with the design of the entire development.

The design of a residential canal development will be a compromise between:

- The objectives of, and economic constraints on, the developer.
- The natural, unalterable conditions at the site, such as climate and the range of the tide.
- Regulations imposed by federal, state, and local agenices.

- The technical limitations imposed by the laws of physics, chemistry, biology, and ecology. These limitations are translated into the ability of the canal system to maintain its physical stability (channel bed and bank geometry), the quality of its waters, and the overall health of the natural systems (vegetation and aquatic life, for example) in its vicinity.

Canal system design, because of its mutual interaction with other facets of the development design, and because of the various constraints listed above, must be iterated with the overall development design. If, for example, the design results in higher water velocities than the slopes of the canal banks can tolerate, the canal design will not pass the canal designer's criteria. If the resulting lot density is not great enough, the design may not meet the developer's criteria. Finally, if the flushing characteristics are not efficient, the canals may have to be redesigned, and this could require a reconsideration of the developer's objectives, the characteristics of the selected site, or the development design itself.

3.3 Analytic and Numerical Approaches

It is often possible to describe and predict the magnitude of a quantity, such as the total flow of water in a channel or the concentration of dissolved oxygen at the surface of the water, by means of algebraic equations which may be solved analytically. There are some spatially - and temporally - varying processes, however, that must be described by equations for which there is no analytic solution for the general case. For example, the equation that describes the concentration of a substance in a moving mass of water as a function of location, time, and the given conditions at the boundaries of the problem is a partial differential equation called the convective-dispersion or mass-transport equation. This particular equation can only be solved analytically for certain limited conditions, and in the general case must be solved by approximation techniques on a digital computer.

The concentration of a pollutant in a tidal canal network may be predicted by the mass-transport equation. The equation is solved at successive computational points, called cells, over successive time steps for a given canal network and specified boundary conditions. The computer program that solves this equation at discrete locations and times in the network is the Hydraulic Laboratory's canal design numerical model CANNET. Since the model predicts the concentration of the pollutant at successive times, the results can be used to determine the flushing effectiveness of the canal network for various network configurations, channel geometries, and pollutant and freshwater inflows.

CHAPTER 4
TIDAL CANAL HYDRODYNAMICS

Materials are removed from a canal network by two different physical phenomena, convection and dispersion. Convection is the transport of a substance by the movement of the water particles, while dispersion causes a substance to spread away from locations of high concentration of that substance. The convective rate, therefore, depends on the magnitude of the current velocity at each location in the canal, and the rate of dispersion depends on the magnitude of the change in concentration (the spatial concentration gradient) and the spatial velocity gradient.

The size and shape (width, depth, and side slope) of the individual canals which constitute a canal network have a major effect on the circulation. These variables may be distinguished collectively by the term "geometry". Fluctuations in the tide, wind, and salinity, as well as the geometry, create the longitudinal, lateral, and vertical convection components that tend to flush tidal canals. These phenomena, of course, vary both with location in the canal network, and with time.

In addition to the convective processes, however, the distribution of substances in a canal network is also influenced by the amount and location of inflow of those substances. Sources of inflow include rainfall and runoff, seepage through the bed and banks, and exchange with the connecting waterbody through tidal entrances.

The local climate has a major influence on the movement and flushing of substances in a canal network. This effect is more difficult to describe than the effects of geometry and tides because it involves variables that fluctuate over much longer time periods. For example, the wind induces a relatively higher water velocity at the surface of the canal, in the direction of the wind, as well as transversely circulating cells, that decrease rapidly with increasing depth and may produce, near the bed, a component in the opposite direction to the surface wind component.

4.1 Turbulent Flow

All movements of water in a tidal canal or canal network fall into the category of turbulent flow. Turbulent flow is distinguished from laminar flow by the presence of random fluctuations in the water particle velocities, and circular motions called turbulent eddies which act in both the horizontal and vertical planes. These random fluctuations and turbulent eddies promote mixing of the water; in fact, together with the exchange of water between the canal network and the receiving waterbody by the tide, this turbulent mixing is the fundamental cause of almost all of the flushing that will occur.

The velocities of the water particles in the three coordinate directions, in turbulent flow, may each be written in terms of a time-mean average and a component fluctuating about this average. If the time-mean is taken over a period long enough to span the period of the longest turbulent fluctuations, but still very much less than the tidal period, then the flow characteristics and the changes in these characteristics over a tidal period may be used to describe the fundamental tidal circulations. The magnitude of the turbulent fluctuations may be accommodated by means of turbulent coefficients in the equations.

When the basic fluid flow equations are written for a particular flow, the equations are in the same form as those for steady (non-time-varying) flow with the understanding that velocities and other time-varying components are to be considered as time-mean values over a short time period.

The flow of water within a particular canal geometry is described by equations derived from the five basic fluid flow equations:

- the conservation of mass equation,
- three components of the conservation of linear momentum equation, one in each coordinate direction (in a cartesian coordinate system)
- the conservation of energy equation.

These equations may be simplified to some extent by the fact that water is essentially incompressible in natural waterbodies.

Conservation of mass. The conservation of mass equation, also referred to as the continuity equation, states that the time-rate-of-change of fluid mass within a specified volume of fluid is equal to the difference between the inflow and outflow of mass per unit time. Thus, the conservation of mass equation may be written:

$$\frac{\partial}{\partial t}\int_{\forall} \rho \; d\forall = -\int_{A} \rho \; \overline{V} \cdot d\overline{A} \qquad (4.1)$$

where t = the time variable, [T]; ρ = mass density of fluid, $[ML^{-3}]$; $\forall$ = volume of fluid, $[L^3]$; $\overline{V}$ = fluid velocity vector, $[LT^{-1}]$; and $\overline{A}$ = area of boundary of volume through which flow into or out of the volume is to be accounted for, $[L^2]$.

The integral on the left hand side of equation (4.1) represents the total mass of fluid within the defined fluid volume, and the partial derivative with respect to time expresses the rate of change of this total mass. The integral on the right hand side represents the net flow across the boundaries (across the surface, A, of the control volume). The negative sign appears because, by convention, flow directed out of the fluid volume is positive.

Conservation of momentum. The conservation of momentum equation may be written either for changes in momentum in essentially linear fluid motion, or for rotary fluid motion. Since the time-averaged flow in tidal canals is predominantly linear, the angular momentum equation is not required. The linear momentum equation states that the change in flux of momentum through a defined fluid volume is equal to the sum of the forces on the fluid volume:

$$\int_A \rho\ (\bar{V} \cdot d\bar{A})\bar{V} = \int_{\forall} d\bar{F} \qquad (4.2)$$

where $\int_{\forall} d\bar{F}$ = the sum of all forces acting on the fluid volume, $[MLT^{-2}$ or $F]$.

The integral on the left hand side of equation (4.2) represents the time-rate-of-change of momentum of the fluid over the surface area of the specified fluid volume. The integral on the right hand side is the sum of all of the forces on the fluid volume. Since the equation is written in terms of velocity vectors, it can be written as three similar equations in terms of the components of momentum and forces in each of the three coordinate directions (x, y, and z). This equation provides a mean for finding unknown fluid forces when the geometry and flow characteristics are specified.

Conservation of energy. The conservation of energy equation relates the transfer of heat and work at the boundaries of a system to the change in internal energy of the system. When applied to an open-channel flow, the terms in the energy equation may be related directly to properties of the flow, and the equation takes the more familiar form:

$$H_1 = H_2 + \Delta H_{LOSSES} \qquad (4.3)$$

where

H = total energy per unit weight at a specified section (number 1 or 2 in this case) in the flow, [L]

$$= z + \frac{p}{\gamma} + \frac{\alpha V^2}{2g} \qquad (4.4)$$

and z = elevation above datum, [L]; p = pressure at elevation z, $[FL^{-2}]$; γ = unit weight of water, $[FL^{-3}]$; α = energy coefficient, or kinetic energy correction factor, [dimensionless]; V = mean velocity of flow, $[LT^{-1}]$ = Q/A; and ΔH_{LOSSES} = energy loss per unit weight inside the fluid, due to friction and turbulence. Equation (4.4) is only valid when the section is plane and normal to streamlines. Streamline curvature must be negligible.

The energy equation provides a means for evaluating friction losses from measurements of flow characteristics at two or more sections in the flow field.

4.2 Energy Sources and Uses in Tidal Canals

The total energy available for mixing, flushing, dispersion, scour and sediment transport in a canal system consists of the potential energy of the tides, wind energy transferred into the water to form waves and impart surface drift and subsurface transport, and the freshwater inflow over control structures or through the canal banks. The wind energy and freshwater flow are seasonally variable with random components, but the tidal energy is more-or-less constant, averaged over the synodical month. The potential energy provided by the tides over one half tidal cycle may be approximated by

$$E_p = 2\, a^2\, \gamma\, A_{ws} \tag{4.5}$$

where a = tidal amplitude, [L]; γ = specific weight of water, [FL^{-3}]; and A_{ws} = water surface area in the entire canal system at mid tide. Thus, at a location where the spring tidal range is twice the neap tide range, four times as much energy is available during spring tides than during neap tides.

The range of tide, and the average power delivered by the tides during the flood, are relatively small in Floridian canal systems. Typical values are compared in Table 4.1. The first entry is for an entire canal system, while the other three are for individual finger canals. It should be noted that the available tidal power, expressed in the familiar units of horsepower, is relatively small even in a large canal system. Also, since the equation for potential energy is a function of the tidal amplitude squared, the available energy decreases rapidly with tidal amplitude.

The fate of this energy depends strongly upon the form (shape and geometry) of the canals. In rivers and streams the "useful" energy is considered to be that which is available for erosion and sediment transportation. However, in a canal system, in which the tidal energy is limited, it is necessary to manage the utilization of that energy so that mixing of the water and flushing of pollutants occurs throughout the canal network.

TABLE 4.1

TOTAL AVERAGE POWER DELIVERED TO WATER IN
VARIOUS FLORIDIAN CANAL SYSTEMS BY TIDE DURING ONE FLOOD TIDE

Name of Canal System	Location	A_{ws} $(ft^2) \times 10^4$	Tidal Range (ft)	Total Ave. Power (HP)
57 Acres Canal Network	ICWW South of Jupiter Inlet	1300	1.25	105
S.E. 14th St. Canal	Pompano Beach	8.77	3.53	5.7
North Canal, Loxahatchee River	4 3/4 miles up Loxahatchee River from Jupiter Inlet	6.47	2.08	1.5
Bangsberg Waterway	Port Charlotte	5.34	2.16	1.3

*ICWW = Intracoastal Waterway

In the more standard civil engineering problems associated with flood projects, hydroelectric projects, drainage canals, and pipe flow, the energy loss due to roughness, viscosity, channel or pipe size and velocity have generally been lumped into single coefficients such as Manning's and Kutter's "n", the Chezy "C" or the Darcy-Weisbach friction factor "f". The "effective roughness", however, is only partially a function of the size, number, and concentration of roughness elements within the system. Other energy losses included in the "effective roughness" are associated with bends, involving internal friction from secondary currents, reduction of effective cross sectional area due to eddies accompanying flow separation downstream from bends, and repeated velocity changes. These "losses" are attributed to expanding or contracting sections. Although certain of the non-boundary-friction losses can be calculated separately,they are generally handled by adjusting the empirically determined friction factors or coefficients in accordance with measured data. To the engineer trying to efficiently move water from one place to another, these are legitimate losses, but to the environmental engineer, whose goal is to preserve and promote water quality in a residential canal system, the "losses" should be divided into two categories:

a) those which result in the direct production of heat without performing useful work, and

b) those which result in mixing and flushing.

Any energy not converted to heat as the tide goes from full high to full low(or vice versa) will show up as eddies in portions of the canal system where they can be sustained because of favorable boundary conditions. One of the goals of canal design is, then, to design these boundary conditions properly.

As in a simple mechanical system, the losses to heat are associated with friction. The friction is directly felt as the water flows past the bed and the banks. Internal friction is a function of the scale and intensity of turbulence which, in turn, is induced by roughness elements of all sizes from the sand grains on the bed to changes in cross sectional area. Neither boundary friction nor internal friction can be eliminated, but the magnitude of the loss, and rate at which the loss occurs, is strongly influenced by form factors in the canal system. With a detailed quantitative knowledge of these influences in relation to tidal prism, a canal system could be specifically designed as an efficient mixing and dispersion system.

4.3 Basic Equations for Tidal Flow in Canals

The equations for tidal flow in an open channel have been derived in detail by Harleman and Lee [1969, pp. 13-21] and others from the conservation of mass and momentum equations.

The conservation of mass equation, in this instance, may be written:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} - q_I = 0 \tag{4.6}$$

where A = cross-sectional area of the channel, $[L^2]$; Q = total rate of flow, $[L^3T^{-1}]$; and q_I = lateral inflow rate per unit length longitudinally along the channel, $[L\ T^{-1}]$.

From equation (4.6) more specific equations for the flow in tidal canals may be derived by limiting the complexity of the cross-sectional geometry. For example, a trapezoidal cross-section permits the channel to be described by a single (but possibly different) bank slope on each side, but not by multiple slopes or storage zones along the principal channel. Thus, the cross-sectional area of a trapezoidal channel is given by

$$A = bd + sd^2 \tag{4.7}$$

where

$$d = d(x,t) = d_0 + a \sin \omega t = \text{depth, } [L],$$

and b = channel width at the bed, [L]; s = inverse side slope, [dimensionless]; d_0 = depth at mean tide level (MTL), [L]; a = tidal amplitude, [L]; ω = tidal frequency, $[T^{-1}] = 2\pi/T$; and T = tidal period, [T].

The expression for the tidal depth, as given above, has been simplified to a harmonic function with only one frequency. This could just as well be expressed by means of a Fourier series to account for the other principal tidal constituents, but the development of analytic expressions for the tidal velocities would then not be straight forward. The inverse side slope is the cotangent of angle θ in the diagram of a trapezoidal channel section (Figure 4.1). Note that the rectangular channel is specified when θ is zero.

The general (one-dimensional) momentum equation for tidal flow in open channels, as derived from equation (4.2) by Harleman and Lee (and others) is

$$\frac{\partial Q}{\partial t} + u \frac{\partial Q}{\partial x} + Q \frac{\partial u}{\partial x} + g \frac{\partial}{\partial x} (hA) + g \frac{Q\,|Q|}{AC^2R} = 0 \tag{4.8}$$

where g = acceleration due to gravity, $[LT^{-2}]$; u = u(x,t) = cross-sectional-mean (longitudinal) velocity, $[LT^{-1}]$; h = h(x,t) = height of water surface above datum, [L] = z + d; C = Chezy's coefficient, $[L^{1/2}T^{-1}]$; and R = hydraulic radius, [L].

4.4 Longitudinal Velocity

The longitudinal velocity component, u(x,t), in a tidal canal varies with time and location in the canal system. Its magnitude is influenced by tide, wind, water density gradients, and irregularities in channel geometry, such as bends. The velocity may be found at any location and time within a tidal cycle by summing expressions for each of the component effects listed above, i.e., tide, wind, density gradients, and bends.

4.4.1 Tidal velocity component. It has been observed that the tidal range along the coast of Florida is comparatively small. These small tidal ranges induce very small velocities in the tidal canal networks; it is not uncommon, for example, to measure tidal velocities of less than 0.5 fps at the tidal entrances. This observation leads to the suggestion that the flow in a canal network might be well approximated by a horizontal water surface rising and falling with the tide. If this approximation is valid, it may be further assumed that the acceleration terms in the momentum equation are negligible. The conservation of mass equation alone is then sufficient to uniquely determine the velocity field [Walton, 1976a]. To test this assumption, the results of tests on a straight, rectangular, 11,000 ft (3350 m) canal using both Harleman and Lee's model and the model based on the horizontal water surface assumption have been compared [Morris, Walton & Christensen, 1978, pp. 145-148]. The computed depths, water surface slopes, and velocities compared to within 2 percent, which was the basis for using the horizontal water surface assumption in the CANNET model.

Expressing the mean depth as a function of tidal amplitude and period,

$$d = d_o + a \sin \omega t \tag{4.9}$$

as used above in conjunction with equation (4.6), introducing this into the conservation of mass equation, and integrating, the tidal velocity may be written

$$u_t(x,t) = \frac{1}{A} [-x \frac{dA}{dt} + \int_o^x q_I \, d_x] \tag{4.10}$$

Note that equation (4.10) states that when there is no lateral inflow, q_I, the longitudinal tidal velocity is a linear function of the distance, x, from the dead end of the canal to the location at which velocity is being calculated.

4.4.2 Vertical tidal velocity profiles. The variation of the longitudinal tidal velocity with depth, and with distance across the canal, influences the magnitude of the dispersion that will result. A convenient approximation in the vertical is a logarithmic profile, which for flow in the turbulent rough range is

$$u_t(z,t) = 2.5u^* \ln 29.7 \frac{z}{k} \tag{4.11}$$

where z is the distance vertically above the bed, [L]; u* = friction or shear velocity, [LT^{-1}]; and k = Nikuradse's equivalent sand roughness, [L]. Although this equation is not defined at the bed, the velocity rapidly approaches zero as z approaches k. The flow will be in the turbulent rough range if the following two criteria are satisfied:

$$Re_* = \frac{u^*k}{\nu} > 70$$
$$Re' = \frac{\bar{u}R}{\nu} > 580 \tag{4.12}$$

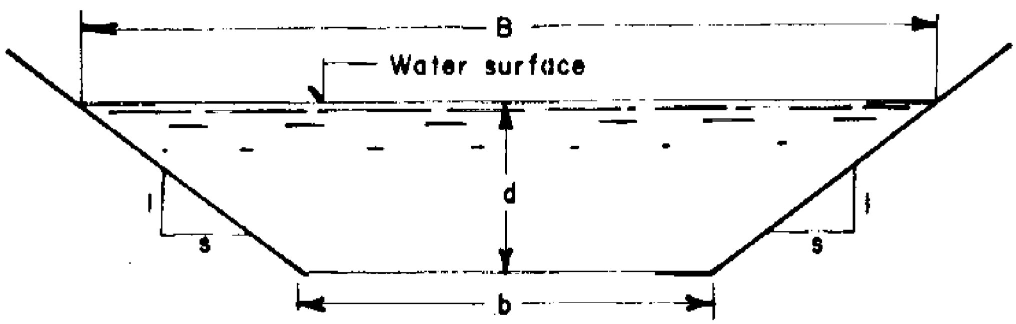


Figure 4.1 - Definitions of Geometry for Trapezoidal Section.

where Re_* is the wall Reynolds number, [dimensionless]; Re' is the Reynolds number based on hydraulic radius, [dimensionless]; $\overline{u}$ is the cross-sectional, time-mean velocity, $[LT^{-1}]$; and ν is the kinematic viscosity of the water, $[L^2T^{-1}]$.

4.4.3 Wind-induced velocity component. Wind blowing over a canal can have a substantial effect on the circulation within the canal. The magnitude of this effect depends upon the magnitude of the surface wind velocity and the direction of the wind relative to the orientation of the canal. Since wind is normally quite avariable in both magnitude and direction, and the individual canals in a network are oriented in a few, or many, different directions, the effect of the wind on the circulation must be evaluated in a real-time sense in conjunction with the tidal fluctuations.

The wind-induced vertical velocity profile caused by that component of the wind that is oriented in the direction of the canal has been derived theoretically by Cooper and Pearce [1977] and others. As shown in Figure 4.2, the water velocity in the surface layer is in the direction of the prevailing wind, and relatively large. If conservation of mass is strictly followed in a dead-end canal operating under the horizontal water surface assumption, a return flow is induced in the lower layers. The theoretical development results in a zero-velocity at the two-thirds depth and a maximum velocity in the direction opposite to the wind at the one-third depth. Measurements of wind-induced circulation effects in tidal canals have confirmed this theoretical profile [Morris, Walton & Christensen, 1978, p. 14].

When superimposed on the logarithmic vertical tidal velocity profile, the canal-oriented component of the wind produces one of the two composite vertical profiles shown in Figure 4.3. If tide and wind are in the same direction, the velocity in the surface layer is relatively large and the return velocity in the lower layer is small. If tide and wind are oriented oppositely, the surface velocities are somewhat smaller and the lower velocities are larger. In addition, due to the mass of the water, these vertical velocity profiles take some time to become established and therefore the wind effect is integrated over a substantial period of time.

It should also be realized that with this model of the wind-induced circulation, there will be a vertical circulation near the dead-end of the canal, whether the wind is oriented toward the dead-end or toward the mouth of the canal. This effect is an indication that there will be increased vertical mixing in the presence of wind. It is true that a prevailing wind toward the dead-end of a canal will cause the accumulation of floating trash, if there is any, at the dead-end. This, however, is no indication that the canal waters are polluted; in fact, the presence of floating trash near a dead-end would be an indicator that mixing and flushing in such a canal is better than in one in which floating trash is distributed more uniformly along its length.

The wind induced vertical velocity profile in a canal may be written in the form

$$u_w(z) = \frac{K_w\, w_s \cos\theta\, |\cos\theta|\, z\, (\frac{3z}{d} - 2)}{4\, N_z} \qquad (4.13)$$

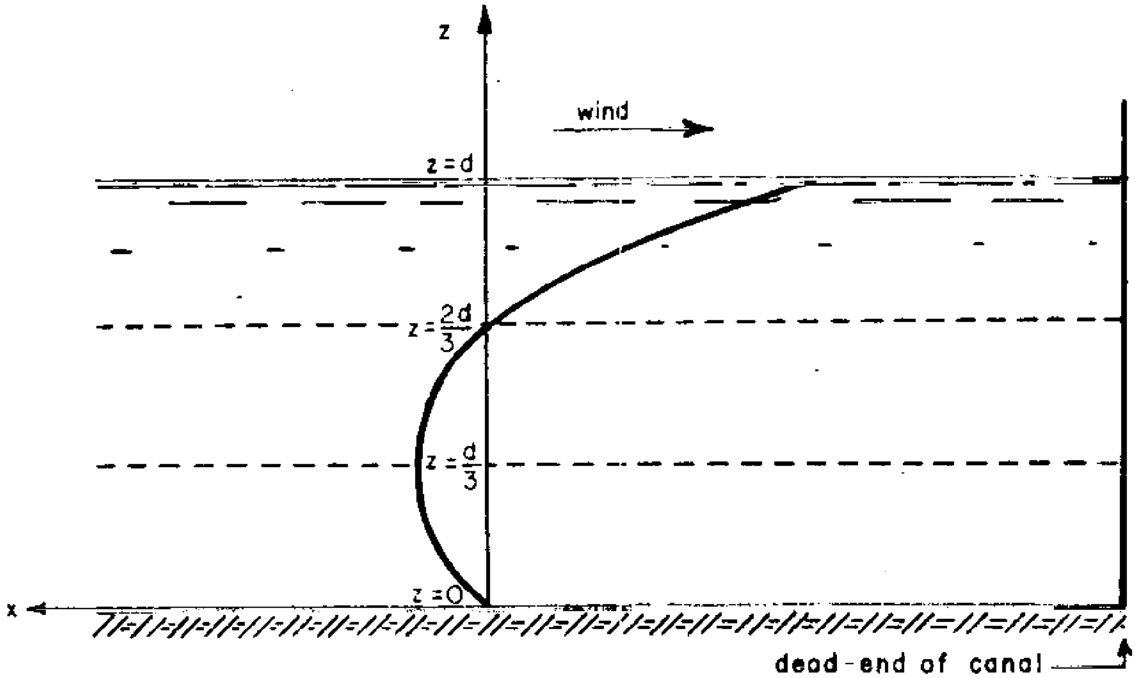


Figure 4.2 - Theoretical Wind Profile.

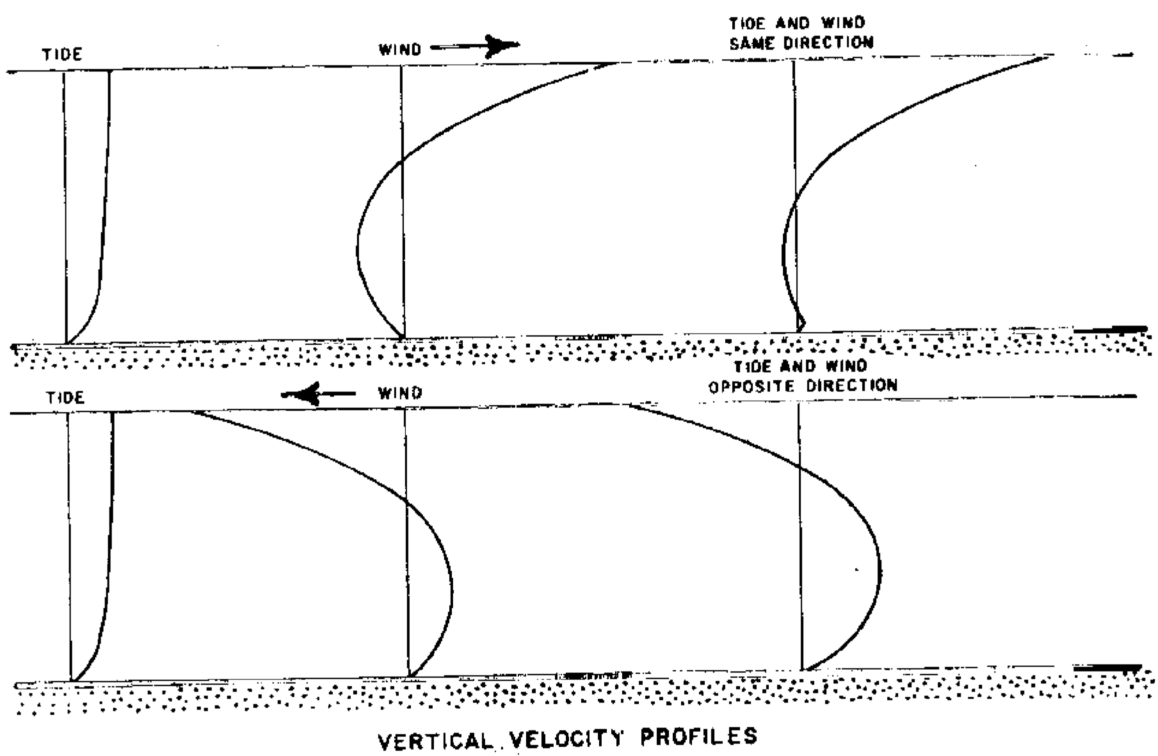


Figure 4.3 - Superimposed Tide and Wind-Induced Velocity Components.

where K_w = wind drag coefficient [dimensionless]; w_s = wind speed, $[LT^{-1}]$; θ = angle between the wind direction and the positive longitudinal direction of the channel, [degrees]; and N_z = vertical momentum transfer coefficient, $[L^2T^{-1}]$. While the classical form of the vertical momentum transfer coefficient, $N_z(d)$, is a parabolic function of bed shear velocity u*, depth z, and total depth d, satisfactory results were achieved by expressing this coefficient as a linear function of depth.

4.5 Secondary Currents

A secondary current is the movement of a mass of water in any direction other than the principal flow direction, such as the flow in turbulent eddies or the flow around obstructions. These currents may occur in both straight and curved sections, and their patterns may fluctuate rapidly or be sustained over long periods of time. They may be caused by irregularities in channel geometry, transverse wind components, fluctuations in discharge, and density gradients. Their effect is to assist in mixing and therefore in reducing the local concentrations of pollutants in the canals.

In bends, a resulting current in the form of a helix is produced by the effect of the centrifugal force exerted on the water particles. On the outside of the bend the higher velocities result in a super-elevation of the water surface, while on the inside of the bend the water surface is lowered. A secondary flow is then created downward on the outside of the bend and upward on the inside of the bend, as shown in Figure 4.4. The superposition

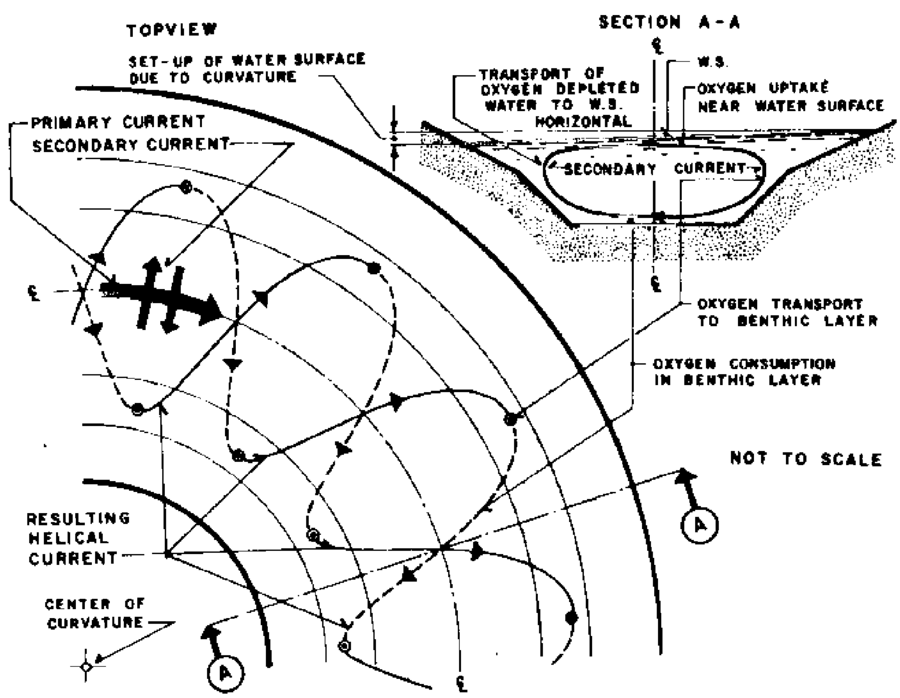


Figure 4.4 - Idealized Helical Flow in a Bend.

of this secondary flow on the primary movement of the water results in the helical water particle path, which persists some distance downstream from the bend. This helical flow is an important canal design element because it enhances vertical mixing. The equations describing the radial velocity component, and the distance downstream over which the radial velocity decays, are developed in Morris, Walton & Christensen [1978, pp. 236-244]. The CANNET3D version of the canal model must be used if the effects of bends are to be included in a design evaluation.

4.6 Stratified Flow

The water in some Floridian canals is well mixed, in terms of density and temperature. In other canals it may not always be homogeneous. When the gradients of temperature and/or density become sufficiently large to affect the circulation in the canal, then the flow may be designated as density- or temperature-induced. When these gradients become so large that two or more distinct layers of water form, one above the other over a significant length of the canals, these conditions may be called "stratified". When stratification occurs, a distinct interface forms between the two water masses, sloping downward and inward into the canal. An example of measurements indicating stratification in a canal is shown in Figure 4.5. In this case, the denser water mass is usually referred to as a "saline wedge".

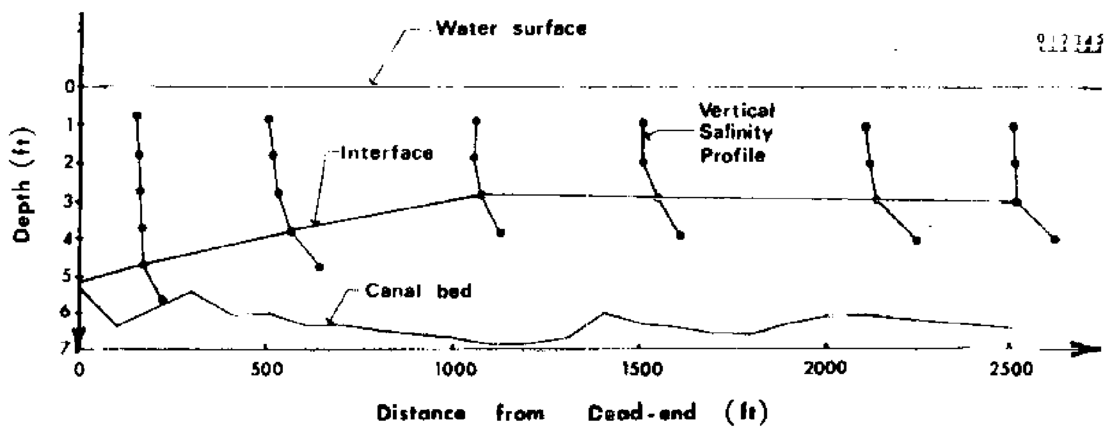


Figure 4.5 - Typical Measured Salinity Profile Showing Presence of Saltwater Wedge.

Salinity gradients and/or stratification commonly occur in Floridian canals during the wet (summer) season due to runoff from rainstorms. Figure 4.6c from Lindall, Fable & Collins [1975, pp. 82-83] shows the change in the salinity difference between surface and bottom stations in a Tampa Bay canal system (Figure 4.6a) during October 1971 and August and September 1972. The maximum salinity difference shown here is 4.5 ppt in October 1971 at station 3. It will be noted that water temperature was close to uniform at all times except in January and February in this set of data (Figure 4.6b) and that dissolved oxygen gradients (Figure 4.6d) tended to form during the summer months when salinity gradients formed. It will also be noted that after the rainy season subsided (October) the canals destratified again.

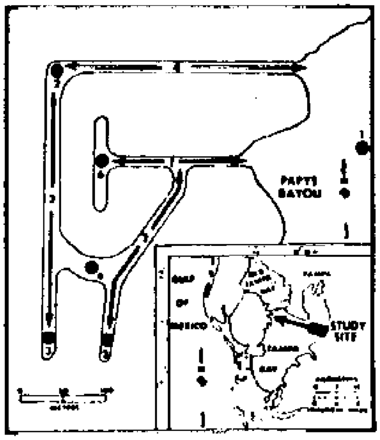


Figure 4.6a - Tampa Bay, Florida, Showing Location of Study Area. Hydrologic Sampling Stations are Shown by Arrows.

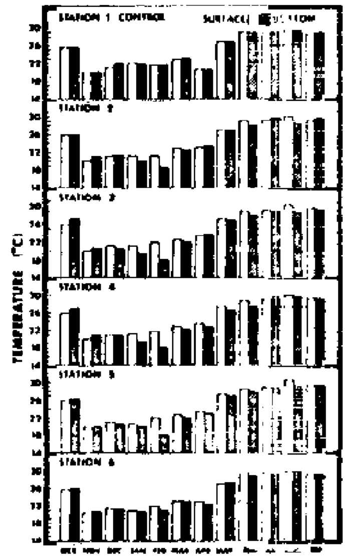


Figure 4.6b - Monthly Water Temperature at the Surface and Bottom of all Hydrologic Stations, Oct. 1971 - Sept. 1972.

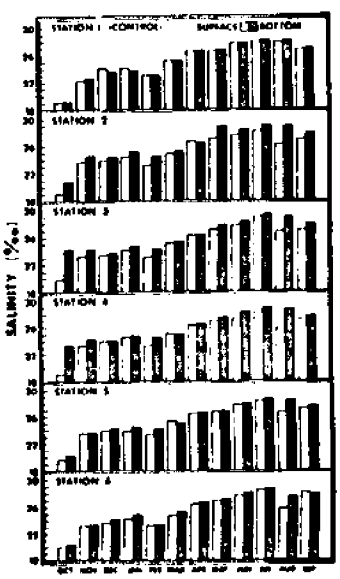


Figure 4.6c - Monthly Salinity at the Surface of all Hydrologic Stations, Oct. 1971 - Sept. 1972.

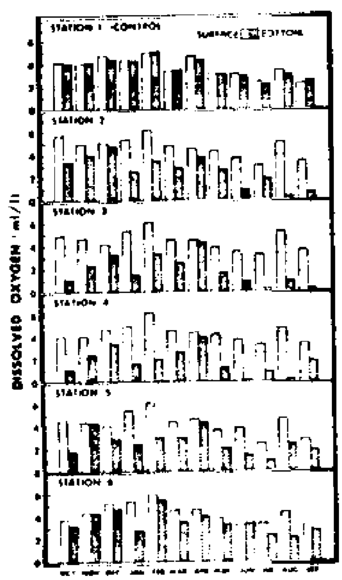


Figure 4.6d - Monthly Dissolved Oxygen at the Surface and Bottom of all Hydrologic Stations, Oct. 1971 - Sept. 1972.

(Source: Lindall, Fable, and Collins, 1975, pp. 82-83.)

Salinity gradients can also occur when fresh river water interacts with saline tidal water near the mouth of a canal, and both are introduced into the canal system on the flood tide. It is also possible for canal waters that are not well flushed to increase their salinity locally through evaporation and form local density gradients which will further inhibit flushing. This might occur, for example, near a dead-end which is located far from the tidal entrance of a canal system, if little circulation is taking place.

The effect of a large density gradient in the vicinity of the salinity interface is to reduce vertical diffusion, which suppresses vertical mixing. If the saline wedge remains in the canal over a period of time, fluctuating in position perhaps but not permitting much of the bottom waters to be exchanged, anoxic conditions can result at the bottom and pollutants could be trapped under the saline layer.

If conditions are favorable for stratification in a particular canal, the saltwater has a tendency to remain together as a unit over many tidal cycles. On a flood tide the saline wedge displaces the lighter, less dense water, temporarily mixing it vertically. On the ebb tide, the elevation of the salt wedge at the tidal entrance falls in response to the change in the elevation of the saltwater in the receiving waters. Due to frictional retardation of the movement of the wedge out of the canal, a saltwater dome is frequently observed in the canal at low tide under these conditions.

The hydrodynamics of salt wedge movement are described in Morris, Walton & Christensen [1978, Section 4.5]. The CANNET3D version of the canal model can simulate the movement of a salt wedge in a canal network, but the results are dependent upon obtaining a value for a coefficient, u_4, from measurements in the canal to be simulated. Since the variability of this coefficient in Floridian canals has not yet been determined, the salt wedge portion of the model cannot yet be used for predictive design of a non-existing canal.

4.7 Friction Effect From Measured Vertical Velocity Profiles

The friction effect is quantified by means of the variable, k, which is Nikuradse's equivalent sand roughness (dimension L). This variable is defined (theoretically) as the size of the roughness elements along the wall of a closed conduit which is completely covered by a surface of these uniform roughness elements. When the flow through such a conduit is steady and uniform, the velocity profile will be logarithmic and the value of k at any cross-section is the diamter of the roughness element.

Extending this concept to unsteady (tidal) flow in a trapezoidal channel, the measured value of k turns out to be on the order of approximately 1 to 20 ft or more due to additional energy losses. This empirical coefficient can be measured if at least the lower part of the velocity profile, near the bed, is logarithmic. This has been shown, by measurements, to be a reasonable assumption for flow unaffected by wind and salinity gradients in Floridian canals, and is justified because at the point where the velocity profile differs from logarithmic the shear stress, τ_0, which is proportional to the velocity gradient, $\frac{\partial u}{\partial y}$, is unaffected by the type of profile above this point. It is only affected by the total depth and the profile close to the bed of the canal.

The relationship between roughness, k, and the logarithmic velocity profile near the bed is given by

$$\frac{u}{u*} = 2.5 \ln(\frac{29.7y}{k}) \tag{4.14}$$

where y = vertical distance from bed, [L]; u = velocity at distance y, [L/T]; u* = bed shear velocity, [L/T]; and k = Nikuradse's equivalent sand roughness, [L].

The bed shear velocity, u*, can be developed from a measured logarithmic profile by selecting two points, 1 and 2, on the profile, with 1 nearest the surface and finding the corresponding velocities, u_1 and u_2. If u_1 is the velocity at $y_1 = p_1 d$ and u_2 is the velocity at $y_2 = p_2 d$, then $u_1 > u_2$ and

$$u* = \frac{u_1 - u_2}{2.5 \ln \frac{p_1}{p_2}} \tag{4.15}$$

Nikuradse's equivalent sand roughness, k, may now be found from equation (4.14),

$$k = \frac{29.7\ p_1 d}{(\frac{p_1}{p_2})^{u_1/(u_1 - u_2)}} \tag{4.16}$$

The bed shear stress, τ_o, is required for analysis of the probability of erosion of bed or bank material. It is given by

$$\tau_o = \rho(u*)^2 \tag{4.17}$$

For example, for the logarithmic velocity profile measured in a straight canal in a system south of Jupiter Inlet, the spatial mean velocity, $\overline{u}$, was 0.153 fps, bed shear velocity, u*, was 0.071 fps, bed shear stress, τ_o, was 0.0098 psf, and roughness, k, was 13.4 ft.

CHAPTER 5
STABLE CHANNEL DESIGN

A flow-through channel, such as a channel connecting two tidal entrances, or a canal reach without a dead-end, should be designed not only to minimize erosion and deposition, but also to accommodate a certain discharge. In general, it is well known that a natural alluvial channel will adapt its depth, width and slope, no matter what the original dimensions are, to a specific set of values for a particular discharge and water/sediment complex. Unfortunately, stable section design in a dead-end residential canal is not practical, because the stable cross-section for such low flows is on the order of a 5 or 10 ft width and a fraction of a foot depth. Such dimensions are frequently seen in natural tidal creeks in Florida. For this reason, an artifical prismatic dead-end canal will always have a tendency toward deposition, and maintenance dredging will be periodically required.

A stable channel, as defined by E. W. Lane [1955] is "an unlined earth canal for carrying water, the banks and bed of which are not scoured by moving water and in which objectionable deposits of sediment do not occur". More specifically, it can be defined as one in which "the time-mean of the local bed shear stress is equal to or less than the local critical shear stress" [Christensen, 1976, p. 2]. If the cross-section of a flow-through channel is designed for tide- and wind-induced velocity for typical climatic conditions, it will still be subject to periodic deposition and erosion. However, over a long period these two phenomena should average out and there will be much less net movement of bed and bank materials than if the canals are not designed for stability.

5.1 Probability of Erosion or Deposition

If the bed or banks of a channel are composed of a cohesionless material, such as fine sand, they will begin to erode when the bottom shear stress, τ_o, due to the velocity of the flow, reaches a maximum value $\tau_{o \cdot max}$. The maximum bottom shear stress is a function of the submerged weight of the topmost layer of grains, the angle of repose (a property of the bed or bank material) and the hydrodynamic lift on the bed or bank area at the moment of incipient motion. Since scour is essentially a stochastic phenomena, a quantity called the critical horizontal shear stress, $\overline{\tau}_{cr \cdot h}$, is defined to permit a probabilistic treatment. The critical horizontal shear stress is the time-mean bed shear stress corresponding to a probability of 1 in 1,000 that $\tau_{o \cdot max}$ will be exceeded. Shields [1936] found that $\overline{\tau}_{cr \cdot h}$ is a function of the unit weight of the material γ_s, the unit weight of water γ, and the equivalent grain size d_e,

$$\overline{\tau}_{cr \cdot h} = A_h(\gamma_s - \gamma)d_e \tag{5.1}$$

where A_h = Shield's entrainment coefficient, [dimensionless], a function of wall Reynolds number; γ_s = unit weight of bed material, $[FL^{-3}]$; γ = unit weight of water, $[FL^{-3}]$ = 64.18 lb/ft^3 for saltwater; d_e = effective grain-size of bed material, [L] (usually expressed in mm) = $\left(\int_{y=0}^{1} \frac{dy}{d}\right)^{-1}$; y = the fraction by weight of the sediment that is finer than d.

For fully turbulent flow (range III), Shield's entrainment coefficient, A_h, is 0.056. The effective grain size, d_e, is defined as the grain-size of a uniform spherical material that behaves in the same way as the natural nonuniform material it represents. It is introduced to enable formulae developed for uniform materials to be applied to cases involving nonuniform materials.

The effective grain-size is obtained from a sediment sample by first thoroughly drying the sample, weighing the total sample, weighing successively smaller samples passed by finer sieves and plotting the fractions in percent finer than each quantity on the ordinate versus grain size on a logarithmic scale on the abcissa (Figure 5.1). The grain size corresponding to the sieve diameter which passes 35 percent of the material, called $d_{35\%}$, is approximately equal to the effective grain size of the sample. A more accurate determination of the effective grain size may be obtained by fitting three or more straight lines to the grain size distribution curve and calculating d_e by the formula,

$$\frac{1}{d_e} = \Sigma_{n=1}^{p} \frac{y_n - y_{n-1}}{\ln \frac{d_n}{d_{n-1}}} \left[\frac{1}{d_{n-1}} - \frac{1}{d_n}\right] \tag{5.2}$$

where p = number of approximating straight lines; d = diameter of grain, L; y = fraction finer than d, by weight.

Erosion will theoretically begin on the bed of a channel when the time-mean bed shear stress $\overline{\tau}_0$ exceeds $\overline{\tau}_{cr \cdot h}$. Measurements of vertical velocity profiles in an existing channel, or predictions of mean velocity from a tidal prism analysis, permit values for $\overline{\tau}_0$ to be calculated.

The critical shear stress on a bank can be related to the critical horizontal shear stress by several methods. For channels which are hydraulically rough the effect of hydrodynamic lift on the sediments may be neglected. The criterion for ignoring the lift is met if the ratio of the equivalent sand roughness, k, to the effective grain size, $d_{35\%}$, is greater than or equal to 100.

From measurements in Floridian tidal canals, it has been shown that the ratio, r, of equivalent sand roughness k, to effective grain size, $d_{35\%}$ is on the order of 9000 or more. These results justify the use of the USBR (Lane's) formula for the critical bank shear stress.,

$$\bar{\tau}_{cr \cdot b} = \cos\theta \sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}}\ \bar{\tau}_{cr \cdot h} \tag{5.3}$$

where θ = bank slope angle, measured from horizontal, [degrees]; ϕ = angle of repose of material, [degrees].

This provides values for $\bar{\tau}_{cr \cdot b}$ on the order of 0.002 lb/ft^2 or less. See Christensen [1971].

The velocities and shear stresses on vegetated banks will be lower than on non-vegetated banks, since the bed shear velocity, u*, is approximately proportional to the mean velocity (the increase in roughness, k, having a minor effect on the proportionality constant for a logarithmic velocity profile) and the bed shear stress is proportional to $(u^*)^2$. Thus, a vegetated bank will have less tendency to erode both from a hydrodynamic viewpoint and due to the additional stabilization provided by the roots of the vegetation. The bank slope design should therefore be based on providing a stable base for revegetation, and a greater critical shear stress than the bed shear stress that would result from the design storm.

5.2 Stable Cross-Section Design

Christensen [1976] has developed a design technique which provides a value for the mean depth, d_0, and the bottom width, b, for a mild-slope trapezoidal channel with cohesionless alluvial or marine deposits, given discharge, Q, bed slope, S_b, inverse bank slope, s, effective grain-size, d_e, equivalent sand roughness, k, and the probability-dependent critical shear stress of the bed material on a horizontal bed, $\bar{\tau}_{cr \cdot h}$. Figure 5.2 is entered with a trial value for the quantity,

$$\frac{d_0}{\left[\frac{Q}{M\sqrt{S_b}}\right]^{3/8}} \tag{5.4}$$

where d_0 = approximate mean depth, [L]; Q = mean discharge, $[L^3T^{-1}]$; $M = \frac{1.49}{n} = \frac{8.24\sqrt{g}}{k^{1/6}}$, $[L^{1/3}T^{-1}]$; n = Manning's n, [dimensionless]; S_b = bed slope, [dimensionless]. At its intersection with the design value for inverse side slope, s, the value of the ratio, b/d_0, is taken from the abcissa. If the resulting value of bottom width, b, is not suitable, the procedure may be interated. The corresponding value for the bank shear stress is, $\tau_0 = \gamma d_0 S_b$. This should be less than

$$\bar{\tau}_{cr \cdot b} = A_h(\gamma_s - \gamma)de\ \frac{\bar{\tau}_{cr \cdot b}}{\bar{\tau}_{cr \cdot h}} \tag{5.5}$$

Various shapes of stable bank profiles designed in this way are shown, for various values of angle of repose and correction factor, by Christensen [1973].

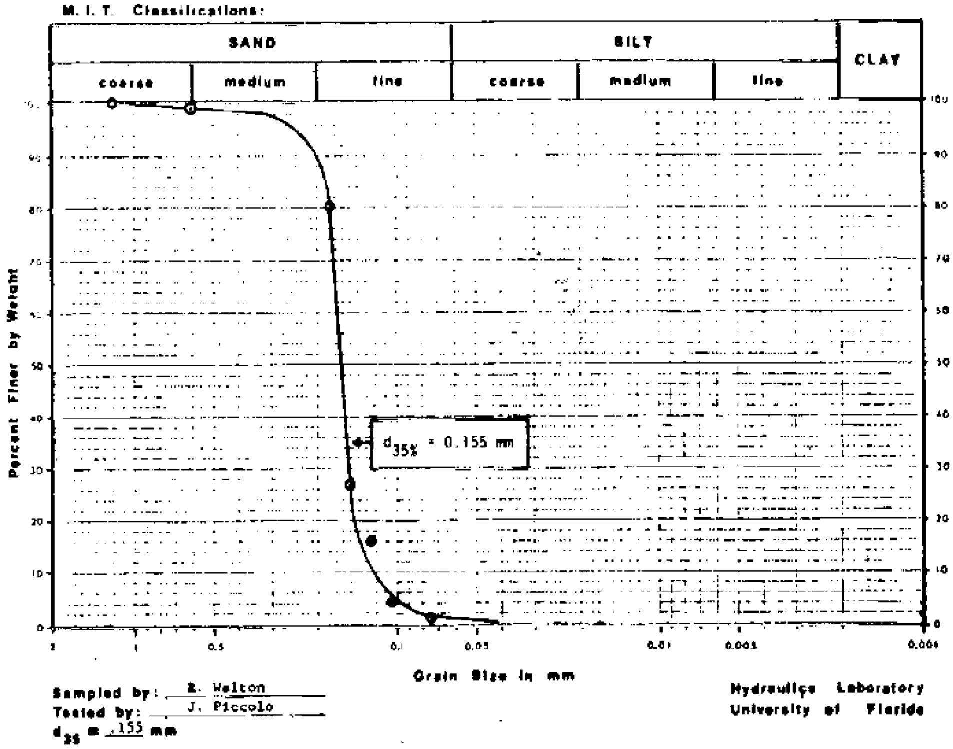


Figure - 5.1 - Plot of Grain Sizes for Determination of Effective Grain Size.

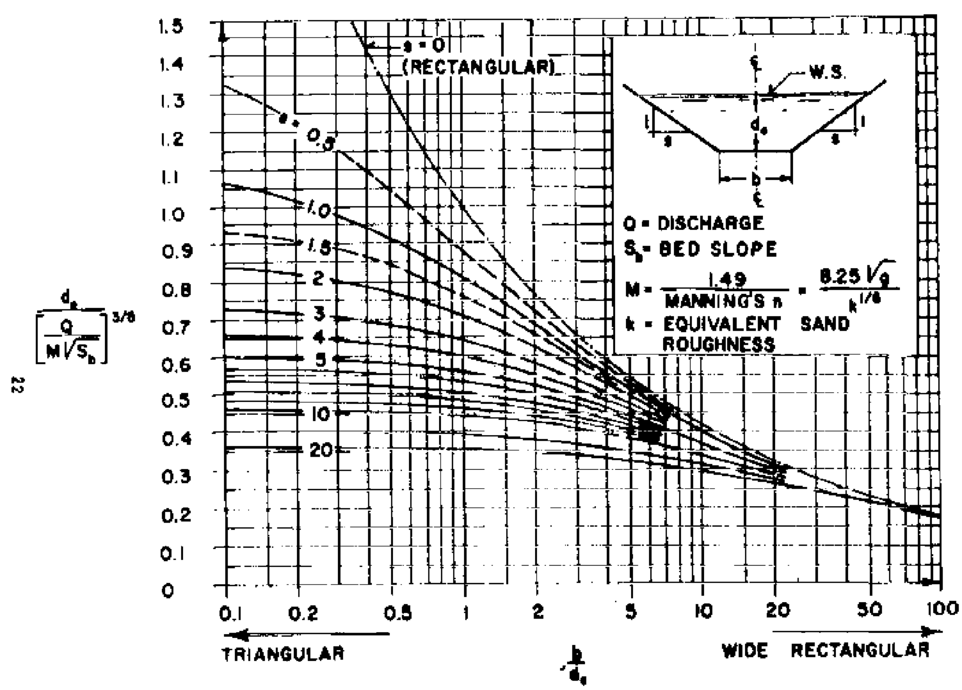


Figure 5.2 - Design Chart for Trapezoidal Channels.

5.3 Bulkheaded and Natural Cross-Sections

A bulkheaded canal has a few features that may seem, at first, to have advantages over sloping banks. As shown in Figure 5.3, the rectangular cross-section is navigable over its entire width, and thus boat handling from the shore may be more convenient than boat handling over shallows. Another convenience is the fact that the bulkhead provides a surface for the attachment of a variety of organisms, which may contribute to the overall health of the canal ecosystem. Still another possible advantage is that, if the bulkheads are constructed and maintained properly, there may be no erosion of bank material, especially as a result of storms.

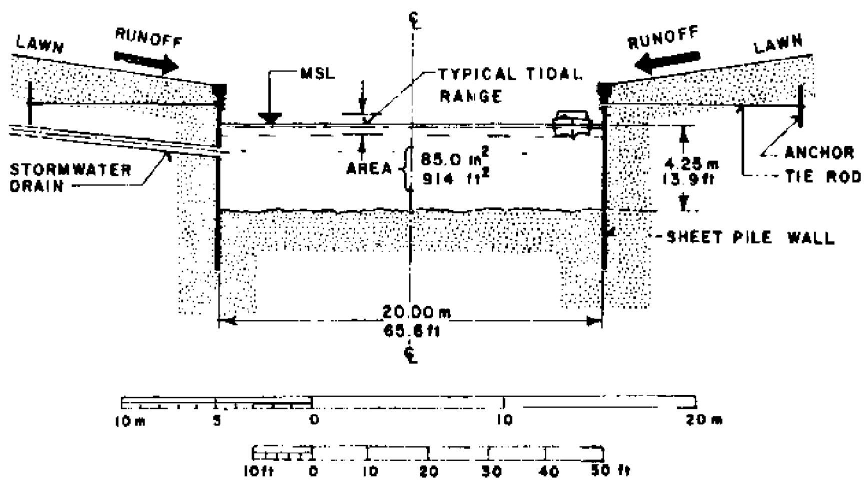


Figure 5.3 - Conventional Bulkheaded Rectangular Canal Section.

The bulkhead, however, cannot stand indefinitely without maintenance. Seepage behind the wall can erode the fill, removing some of its support; tie rods can rust and break; the fill at the toe can erode; or the fill behind the wall can shift (piping), causing the wall to bow and collapse.

A canal with mild-sloped sides (Figure 5.4) provides important advantages in comparison to the bulkhead canal, considered over more than just a few years. If the channel is designed properly for stability, the beds and banks will not have much tendency to erode. Grasses and mangroves will provide additional stabilization against the transient erosion of boat wakes and storm flows. In addition, vegetated banks provide habitat for juveniles of many aquatic species, and uptake of nutrients.

The cross-sectional area of the trapezoidal channel, Figure 5.4, is the same as the cross-sectional area of the rectangular channel, to provide a basis for comparison. It is apparent that the width of the channel is doubled, and the depth is decreased by about 30 percent. These two factors alone make it more difficult to incorporate trapezoidal channels into a canal design, because they require double the land area and because they limit, to some degree, the depth for navigation. However, the properly designed trapezoidal cross-section is self-maintaining and will assist, through its vegetative and aquatic communities, in maintaining water quality. In addition, the cost of constructing vegetated sloping banks is a factor of ten less than the cost of constructing bulkheaded shorelines.

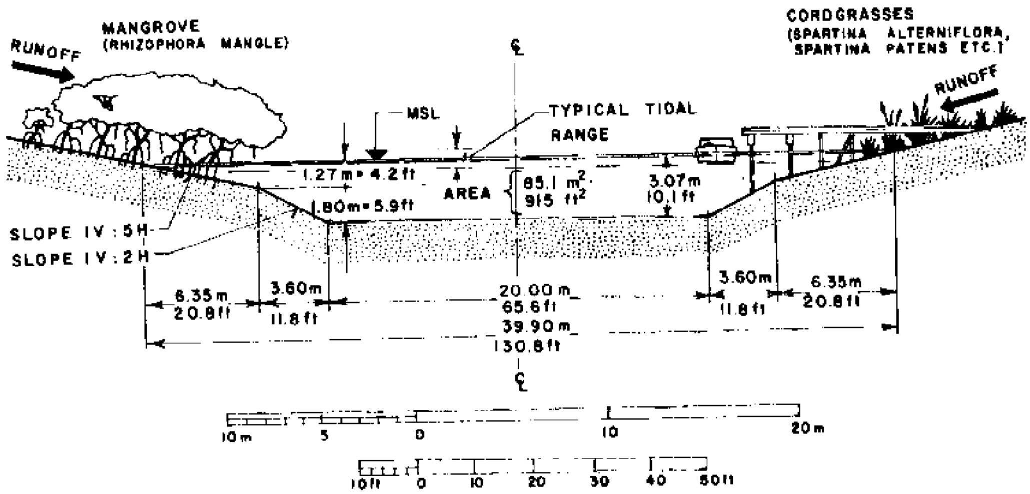


Figure 5.4 - Vegetated Sloping Bank Trapezoidal Canal Section.

If a minimum length of shoreline must be bulkheaded to provide for docking deep-draft boats at each lot, then an average of about 30 feet of bulkhead is required for each lot; if lots have 200 foot waterfronts, 85 percent of the canal lengths may still be constructed with sloping banks. In this situation, it is important to design the transitions from bulkheads to sloping banks to be gradual.

5.4 Meandering Banks

Due to the additional vertical mixing produced by helical flows in bends, it is recommended that curves be designed into canal channels. An example of a meandering channel and its associated lot plan is shown in Figure 5.5. The layout provides one lot for each cycle of the meander, and the dwelling and dock on each side of the channel is faced by natural vegetation on the other side. Thus, the channel will have a natural appearance when viewed either from the water or from the land.

The concept of the meandering bank may be incorporated into a channel in a different manner that may be used for the improvement of existing rectangular channels as well. Instead of designing the bank with curved sides, artificial roughness elements may be placed alternately along the sides of the channel as shown in Figure 5.6. This causes the primary part of the flow to meander, and secondary currents to be set up to increase vertical mixing. A similar effect will occur if, instead of the abrupt roughness elements shown in Figure 5.6, rounded roughness elements are constructed in the bed of the canal as shown in Figure 5.7. The difference between the two methods illustrated in Figure 5.6 and 5.7 is that with the former some energy losses would be experienced, as compared with negligible energy losses with the latter.

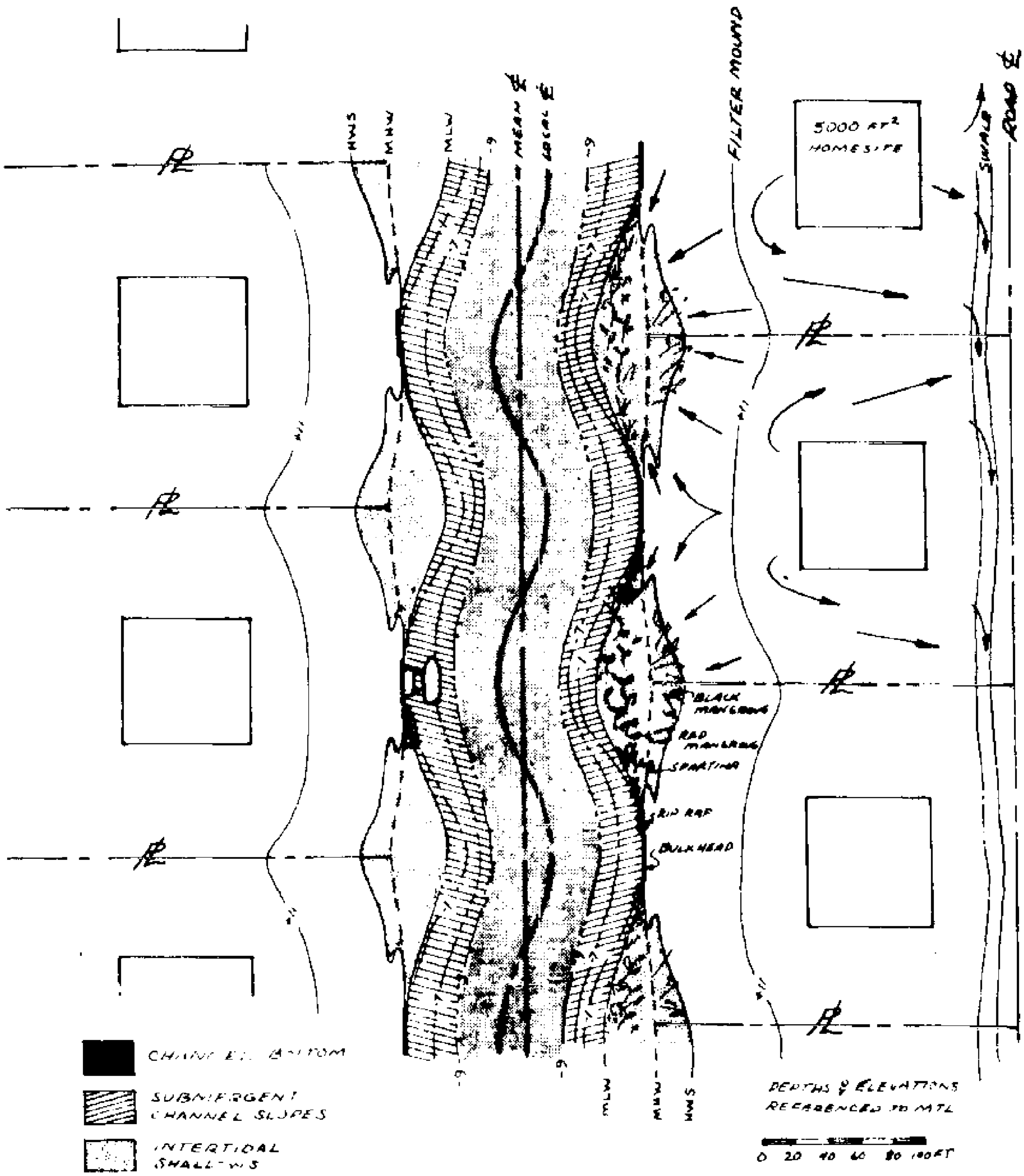


Figure 5.5 - Meandering Bank Design.

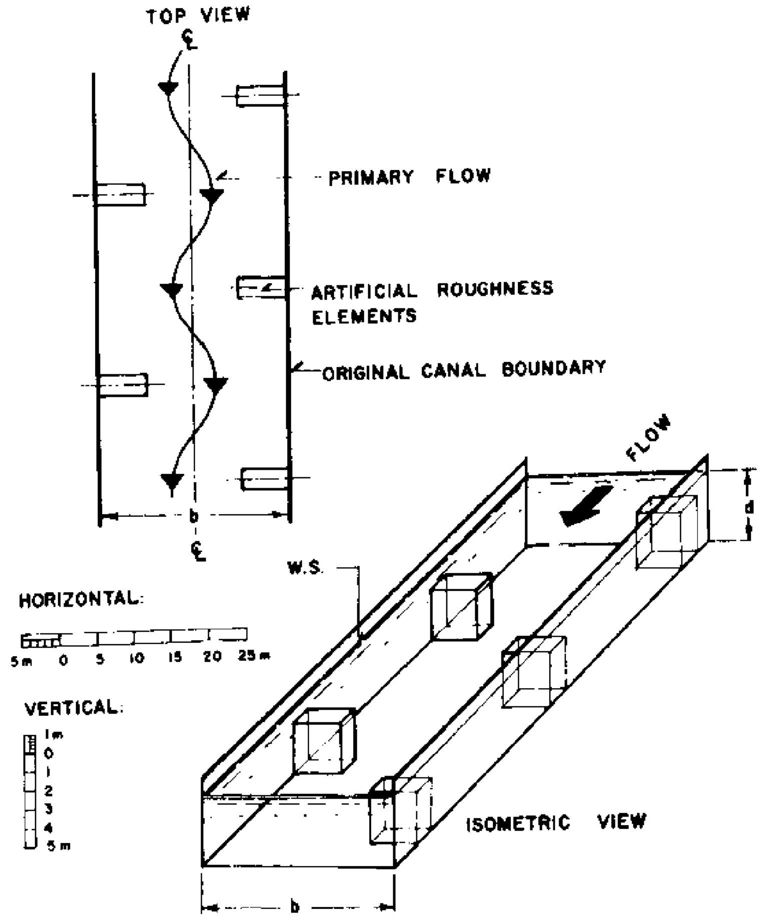


Figure 5.6 - Placement of Artifical Roughness Elements.

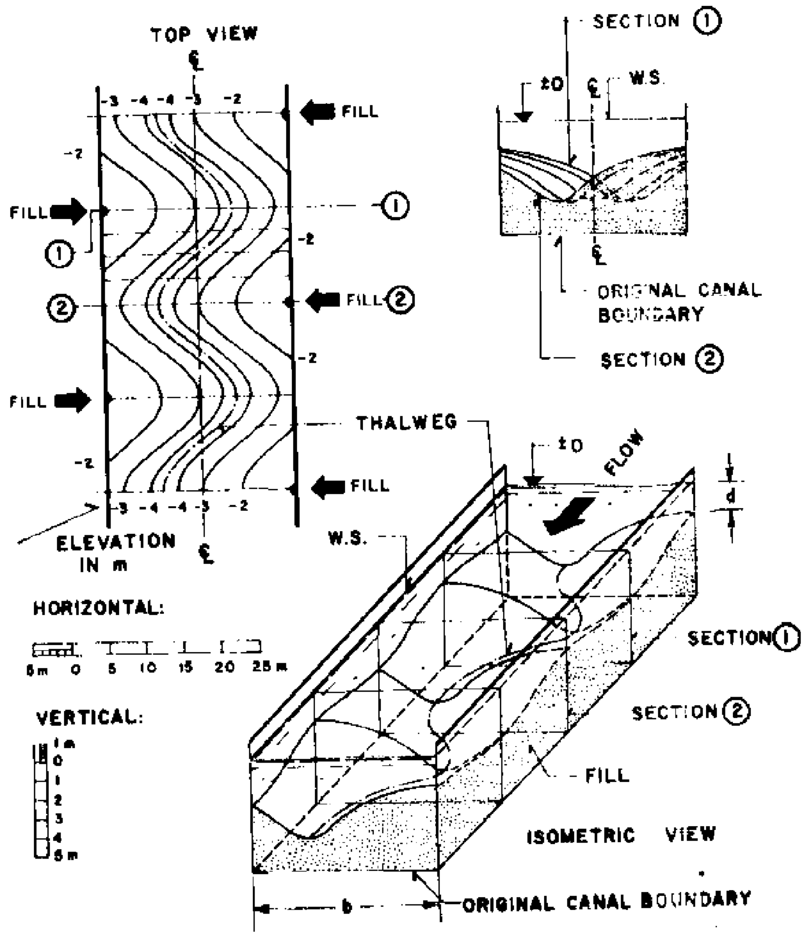


Figure 5.7 - Placement of Fill to Create Meandering Flow.

CHAPTER 6
DISPERSION MODEL FOR CANALS AND CANAL NETWORKS

The dispersion of a substance in a tidal canal or canal network may be described by the hydrodynamic equations and the three-dimensional convective-diffusion equation or mass-transport equation. Since a canal network is often comprised of long, straight, uniform-width reaches, the primary concern of the canal designer will usually be the distribution of a pollutant as it changes with location in the canal network and in depth; the distribution across the channel is usually of little concern. Therefore, a two-dimensional version of the mass-transport equation is normally adequate for canal design, and is much less expensive to solve on a computer than the three-dimensional version.

The partial-differential mass-transport equation applied to canals and canal networks can only be solved numerically. The two- and three-dimensional versions of the canal network model, CANNET2D and CANNET3D, basically consist of a procedure for characterizing the geometry of the canal or canal network, calculating the hydrodynamic equation solutions in time, providing values for coefficients, which may vary in time, solving the mass-transport equation in time, and presenting solutions in a useful format. Since the process of solving the equations in successive time steps appears to reproduce the movement of the pollutant mass, in terms of its concentration at each point in the canal network, a solution in time is often called a *simulation*.

6.1 The Mass-Transport Equation

The three-dimensional mass-transport equation was developed [Harleman, 1966, pp. 576-578; Dailey and Harleman, 1966, Section 16-2.2; Pritchard, 1971, p. 16; Ward and Espey, 1971, p. 17] by considering the conservation of mass of a conservative substance in an elemental volume of the flow. Using the Reynolds analogy for expressing the mass flux per unit area, combined with the Boussinesq assumption in which transport due to turbulent fluctuations is assumed proportional to the time-averaged concentration gradient multiplied by a mixing coefficient, and applying Fick's first law for molecular diffusion, the following form may be developed:

$$\frac{\partial c}{\partial t} + \frac{\partial}{\partial x}(cu) + \frac{\partial}{\partial y}(cv) + \frac{\partial}{\partial z}(cw) = \frac{\partial}{\partial x}(E_x\frac{\partial c}{\partial x}) + \frac{\partial}{\partial y}(E_y\frac{\partial c}{\partial y}) + \frac{\partial}{\partial z}(E_z\frac{\partial c}{\partial z}) + r_p \tag{6.1}$$

where c = concentration of pollutant mass, [dimensionless]; x,y,z = coordinate directions, [L]; u,v,w = velocity components in the x,y,z directions, (respectively), $[L^2T^{-1}]$; E_x,E_y,E_z = turbulent diffusion coefficients in the x,y,z directions, (respectively), $[L^2T^{-1}]$; and r_p = rate of addition or loss of a substance (e.g., a pollutant), $[MM^{-1}T^{-1}]$.

Since the numerical solution of this equation is carried out in discrete time steps, the diffusion coefficients, E, must represent a time-mean change in the distribution of concentration of a substance over one time step in the model. In addition, the diffusion coefficients are used to compensate for unknowns or small-scale fluctuations in the velocity field. Thus, the model must be "calibrated" by adjusting these coefficients until the results match the results of field measurements in a particular canal network.

The rate of addition or loss term for a passive, conservative substance represents the rate of inflow of the substance. If the concentration of the lateral inflow is c_I, then

$$r_p = \frac{q_I \, c_I}{A} \tag{6.2}$$

where q_I = inflow of water containing the substance per unit length along the canal, $[L^3T^{-1}L^{-1}]$; and A = cross-sectional area, $[L^2]$. This equation may be substituted directly into equation (6.1).

6.2 Diffusion and Dispersion Coefficients

A *diffusion* coefficient represents the rate of spreading averaged over a time period associated with the transport mechanism, such as a molecular time scale or a turbulent time scale. A *dispersion* coefficient represents the rate of spreading average over a length scale associated with the transport mechanism. For example, if the three-dimensional mass-transport equation (6.1) is cross-sectionally averaged

$$\frac{\partial}{\partial t}(Ac) + \frac{\partial}{\partial x}(Auc) = \frac{\partial}{\partial x}(AE_x \frac{\partial c}{\partial x}) + Ar_p \tag{6.3}$$

where A = cross-sectional area, $[L^2]$, the coefficient E_x may be replaced by the longitudinal dispersion coefficient, E_ℓ, which by far exceeds the molecular diffusion coefficient in most open channel flow.

The longitudinal dispersion coefficient has been studied from both the theoretical and the empirical viewpoint since the 1950's, when Taylor [1954] measured dispersion in pipe flow, followed by Aris [1956] and Elder [1959] with experiments on flow over a flat bed. Later, Fisher [1967] recognized that this coefficient was dependent to a large extent on the magnitude of the transverse velocity gradient. The resulting form of the longitudinal dispersion coefficient has been expressed as

$$E_\ell = K\,R\,u^* \tag{6.4}$$

where K = dimensionless longitudinal dispersion coefficient; R = hydraulic radius of the channel, [L]; and u* = bed shear velocity, $[LT^{-1}]$. If the vertical distribution of the longitudinal tidal velocity component is assumed to be logarithmic, the spatial mean velocity in the vertical with depth d may be written

$$u_m = 2.5\ u^*\ \ln(10.9\ d/k) \tag{6.5}$$

Equation (6.5) may be solved for u* and introduced in equation (6.4).

The dimensionless dispersion coefficient K is an empirical coefficient which is independent of the depth and roughness of the channel, but a function of the regularity of the channel; low values of K correspond to straight regular canals, while high K values represent irregular curved and meandering systems. Many experiments have been conducted in rivers [Nordin and Sabol, 1974] to obtain data from which the longitudinal dispersion coefficient may be calculated, but few in canals. Measurements by the Hydraulic Laboratory, University of Florida [Morris, Walton & Christensen, 1978, p. 139] provide a range for K of between 2.5 and 41.2 in two canals on the Intracoastal Waterway south of Jupiter Inlet. These values are considerably smaller than those found representative of rivers, since rivers have substantially greater transverse velocity gradients.

CHAPTER 7
FRESHWATER AND POLLUTANT INFLOWS

A canal network will have sources of freshwater and contaminated water. The possible sources of inflow include the tidal entrance, the banks and dead-ends of the canals, the aquifer, and the atmosphere. These inflow sources cannot automatically be categorized either as fresh or contaminated water sources, as pollutants can occur in significant quantities in each.

Pollution is often defined indirectly by various federal, state and local regulations. For example the State of Florida has published a set of criteria for five classes of water [Chapter 17-3, Florida Statutes, Pollution of Waters, Amended 7/3/73; Morris, Walton & Christensen, 1978, pp. 429-430]. These criteria are expressed directly in terms of permitted levels of concentrations of wastes, pH, bacteriological species, toxic substances, deleterious materials, and turbidity, and indirectly in terms of dissolved oxygen. The concentration of dissolved oxygen, in turn, is affected by a variety of physical and biological processes.

7.1 Sources and Effects of Pollution

Not all pollution is a product of man or man-made sources. Natural pollutants, for example, are introduced into canals by natural phenomena of the meterological cycle. Rain, seepage and runoff carry dissolved gases which are native to the atmosphere, and organic and inorganic particles lying on the surface of the earth, into the canals. Table 7.1 summarizes the principal pollution sources and the types of constituents which may be found in residential canal systems. Note that in the category of meterological water, some of these inputs may be spatially distributed, as opposed to point sources.

Pollutants affect the quality of waters in a canal in a variety of ways. *Organics* decay naturally, and their components enter into complex biological and chemical interactions which have a variety of effects on the ecosystem and the transient water quality. *Inorganics* are either transported in suspension or deposited in a sediment layer over the bottom of the canal, and are subject to scour and resuspension when discharge or vertical water movements become significant. Each of these categories may further be broken down into constituents which have to be considered separately if their movement and interactions are to be predicted. Table 7.2 summarizes the harmful effects of pollutants in canal waters.

Water flowing into a canal network from an upland drainage area, a tributary, or the aquifer may often be designated as freshwater. There can be no guarantee, however, that any of these sources, even the aquifer, is uncontaminated.

Table 7.1 - Principal Pollution Soruces for Canal Systems.

Contributing Factor	Components	Principal Quality Input to Surface Waters
Meteorological water	rain runoff seepage flow	Dissolved gases native to atmosphere Soluble gases from man's industrial activities Particulate matter from industrial stacks, dust, and radioactive particles Material washed from surface of earth, e.g.: Organic matter such as leaves, grass, and other vegetation in all stages of biodegradation Bacteria associated with surface debris (including intestinal organisms) Clay, silt, and other mineral particles Insecticide and herbicide residues
Domestic use (exclusive of industrial)	runoff outfall seepage flow	Undecomposed organic matter, such as garbage, grease, etc. Partially degraded organic matter such as raw wastes from human bodies Combination of above two after biodegradation to various degrees of sewage treatment Bacteria (including pathogens), viruses, worm eggs Grit from soil washings, eggshells, ground bone, etc. Miscellaneous organic solids e.g., paper, rags, plastics, and synthetic materials Detergents
Industrial use	outfall seepage flow	Biodegradable organic matter having a wide range of oxygen demand Inorganic solids, mineral residues Chemical residues ranging from simple acids and alkalies to those of highly complex molecular structure Metal ions
Agricultural use	runoff	Increased concentration of salts and ions Fertilizer residues Insecticide and herbicide residues Silt and soil particles Organic debris, e.g., crop residues
Consumptive use		Increased concentration of suspended and dissolved Solids by loss of water to atmosphere

Table 7.2 - Harmful Effects of Pollutants on Canal Waters and Environment.

Type of Material	Effect
Biodegradable organic matter	Deoxygenate water; kill fish, cause objectionable odors.
Suspended matter	Deposit on canal bed: if organic, may putrify and float masses to surface by gas; blanket bottom and interfere with fish spawning or disrupt food chain.
Corrosive substances (e.g., cyanides, phenols, metal ions)	May kill fish and other aquatic life; destroy bacteria and so interrupt self-purification process.
Pathogenic micro-organisms	Sewage may carry pathogens.
Substances causing turbidity, temperature, color, odor, etc.	Temperature rise may injure fish; color, odor, turbidity may render water aesthetically unacceptable for public use.
Substances or factors which upset biological balance	May cause excessive growth of algae or aquatic plants which choke canal, cause odors, etc.
Mineral constituents	Increase hardness, limit use without special treatment, increase salt content to level deleterious to fish or vegation, lead to eutrophication of water.

Source: Adapted from Klein, 1962, in McGauchey, 1968, p. 50.

7.2 Residential Water Use

Estimates of residential water use provide one of the inputs needed for predicting pollutant loading rates into canals via septic tank leaching and overland flow. Water use data are statistical, usually relating a rate in terms of volume/capita-day to population densities, lot size, type of subdivision or a specific municipal or geographic area. Some average residential water use and sewage production data are shown in Table 7.3.

The volume of domestic sewage is given as 80 percent of water consumption in Table 7.3. Goodman and Foster [1969 in Eckenfelder, 1970, p. 60] give lower total volumes of sewage production than in Table 7.3, varying from 70 to 100 gal/c-d depending on the type of housing (Table 7.4). A set of measurements of the characteristics of municipal sewage has shown that BOD accounts for about 10 percent of the total mean concentrations [Hunter and Henkelekian, 1965, in Eckenfelder, W.W., Jr., 1970, p. 58].

7.3 Septic Tanks

EPA has documented extensive studies on septic tank leaching in residential canal developments [EPA, 1975a, pp. 159-186]. In this report on finger-fill canal studies it is observed that septic tank absorption fields are acceptable in rural communities, with long distances to surface waterbodies and relatively low housing density, but that in coastal communities the high density of housing and close proximity to surface waterbodies cause serious leaching problems [EPA, 1975a, p. 9]. The report continues with the following statements:

> It has been indicated that the movement of contaminants through at least 100 feet of unsaturated soil is necessary for effective cleansing in those areas where the groundwater is subject to exchange with surface waters, and that in general, no seepage field be located closer than 300 feet to a channel or water course [Leopold, L.B., 1968, in EPA, 1975a, p. 178].
>
> Even with such a minimum setback requirement, dissolved nutrients may still reach waterways and constitute the potential for creating a biotic imbalance [EPA, 1975a, pp. 178-179].

Clark [1977, p. 503], on the other hand recommends that the absorption field "should be set back at least 150 feet (46 meters) from the annual high-water line".

In general, it would seem reasonable to conclude that septic tank systems are preferable *provided* water and soil characteristics are right and *provided* absorption fields are located and built properly with adequate capacity. Onsite central treatment plants not only have a higher capital cost and higher operating expenses, but also present a potential major pollution hazard in the event of failure in a critical part of the system. More detailed discussions on the environmental effects and design of waste water treatment facilities may be found in Clark [1977, pp. 502-527], EPA [1975a, pp. 3, 9,

Table 7.3 - Average Residential Water User Characteristics.

Gross Lot Size (acre)	Persons per Gross acre	Water Usage GPD	Sewage GPD	Average Sewage gal/cap-day
> 5	< 1	< 125	< 100	< 100
2 - 5	1 - 2	125 - 250	100 - 200	100
1 - 2	2 - 4	250 - 500	200 - 400	100
1/2 - 1	4 - 6	500 - 1000	400 - 800	120
1/4 - 1/2	6 - 12	1000 - 2000	800 - 1600	133

Source: Clark, 1977, p. 823.

Table 7.4 - Domestic Sewage Volume and BOD.

Type	Volume gal/cap-day	lb/cap-day	grams/cap-day
Luxury Homes	100	0.20	90.7
Better subdivisions	90	0.20	90.7
Average subdivisions	80	0.17	77.2
Low-cost housing	70	0.17	77.2

Source: Goodman and Foster, 1969, in Eckenfelder, W.W., Jr., 1970, p. 60.

159-186], and the U.S. Public Health Service's *Manual of Septic Tank Practice*, [1967].

Offsite sewage treatment, or the installation of a package treatment plant, in combination with discharge regulations, prevents the direct pollution of canal and receiving waters by domestic sewage. Use of offsite facilities for sewage treatment moves the problem to another jurisdiction where it may, or may not be handled properly. The adequacy of the offsite facility should be investigated. Package plants can have both positive and negative effects. On the negative side, the disposal of the effluent can still pollute the environment through malfunction, faulty operation or neglect. Thorough studies should be conducted of the proposed disposal method including groundwater flow, appropriate storm statistics and basin hydrographs. On the positive side, the increased use of package plants (mostly on an interim basis) has led to design and construction improvements that obviate problems with earlier units. A properly designed and operated package plant will enhance the local groundwater resource, helping to stem saltwater intrusion.

7.4 Runoff

Rain storms create problems with runoff from roofs, parking lots, roads and lawns. In general, some provision for handling runoff needs to be provided in every residential canal system. Runoff may be controlled by utilizing the natural topography of the site, supplemented where necessary with drainage channels, swales, and detention and/or retention basins, all sized to work together as a system to handle a design storm.

The concept of detention and retention basins has only received significant attention in the United States since the 1960's, although it has been in use in Europe for many years. A *detention* basin is simply a temporary storage location for water, providing outflow rates which are controlled by design. A *retention* basin is a basin designated as a storage volume, which is located so that it will empty by infiltration and evaporation. By analysis of data on the frequency and intensity of rain storms, a hydrograph may be developed and routed through the drainage area on a site to determine the necessary design parameters for an effective drainage plan. Information on the design of hydrographs is summarized in Morris, Walton & Christensen [1978, pp. 578-581].

The best solution to the problem of controlling runoff is usually onsite handling. In this way the rainwater can be infiltrated immediately back into the aquifer receiving effective, free filtering by the soil in the process. One of the costs of providing detention is the commitment of several lot-sized areas to this function, but this area can also be used as a park or recreation area during normal conditions. The alternative is to provide the necessary drainage systems, collection facilities, pipes and pumps for moving stormwater offsite. Stormwater and waste water systems can also be combined, although this is an inefficient and costly way to recycle water.

7.5 Boats and Marinas

The effects of operation of outboard engines on the quality of water and the life systems in a canal would be negligible even if the canal were completely filled with boats, according to information from EPA-funded research [EPA, 1974]. The operation of inboard engines is considered even less polluting, because the four-cycle type of engine burns more cleanly than the two-cycle type. However, boating activity can cause major pollution if people are permitted to dump human wastes overboard, or if a marina does not adequately control its wastes.

The principal problems associated with marinas in the past have been poor location, which results in inadequate flushing, altered water circulation and stagnant pollutant sinks, unacceptable alterations to banks and shoreside vegetation, poor control of drainage from urban structures on shore, release of pollutants such as gas and oil from marina supply sources, and leaching of copper from boat antifouling paint. Flushing can be improved by provisions for adequate flow through the area, shoreline alterations can be made beneficial through proper design, and pollutants can be limited by not providing fueling facilities and not permitting dumping of wastes overboard.

CHAPTER 8
CANAL DESIGN OBJECTIVES, GUIDELINES, CRITERIA AND CONSTRAINTS

A residential canal design is a set of engineering specifications describing the desired appearance and operational characteristics of the proposed canal network. This includes not only specifications for the canals themselves, but any features of the development which will affect the canals. The design provides sufficient detail to enable the detailed construction specifications pertaining to the canals to be drawn up by the developer's engineers, and, therefore, must provide:

- canal layout and geometry
- canal flows
- predicted response of the canal system or network to realistic pollutant inflows
- drainage plan sufficient to eliminate predicted runoff from entering the canals
- geometry and hydrodynamic features of any boat basin to be incorporated into the design
- other pertinent details, such as requirements for vegetation on beds and banks.

The *canal* design supplements the overall *development* design, which is concerned with lot size and placement, roads and bridges, water supply and waste water handling and treatment facilties, utilities, and other common features of residential developments.

In order to provide a logical structure which may be used to discuss the design process, the factors that must be considered by the owner/developer may be divided into four categories:

<u>Design Considerations</u>

Design Guidelines → { Design Objectives ↓ Design Criteria } ← Design Constraints

In this structure, overall initial design objectives lead to design criteria. These are influenced by guidelines which have been developed as a result of analyses of previous developments, and constraints which are either inherent in the site, have been established by the legislative process, or are natural limitations to site and canal design. Examples of considerations in each of these categories are given in Table 8-1.

8.1 Canal Design Objectives

The objectives for a residential canal development are the owner's and developer's concepts of the character, magnitude, and general features which are to be developed at a given site. For example, one objective might be to create a community of quarter-acre lots concentrated along the banks of a principal canal and numerous branching finger canals. Alternatively, an objective could be to create a community consisting of clusters of half-acre lots, interspersed with green spaces, along the banks of a major looping canal with irregular branches arranged to fit into the natural topography of the site. A different type of design objective could be a specified time span from acquisition of the site to completion and sale of the units, which relates to the length of time over which the investment capital can be tied-up in preparation of the site.

The process of fitting a development involving substantial construction into the natural environment so that the two function together harmoniously requires a great deal of planning and insight. To do it properly, the planner cannot ignore any of the many factors which influence and constrain the problem: the natural characteristics of available sites, the land-use guidelines which have already been established in the area, the restrictions imposed by state, federal and local government to maintain a certain minimum quality of environment, the characteristics of the market, and the potential effect of the development beyond the confines of the selected site, to name but a few.

The characteristics of the site limit the canal design plan in certain ways, but also may provide unique opportunities. The design objectives provide a starting point for the canal designer, who will develop a quantitative set of design criteria from them that can be evaluated objectively. It is possible that the designer will discover that the design objectives are not realistic, or are too conservative, after some initial or detailed analyses have been completed. However, if a planning process has been established from the outset, it will be possible to change the plan in an orderly manner as more information is obtained. In Table 8.2 some examples of canal design objectives are presented with the decisions to be made to define these objectives.

The overall objective of the planning process is to avoid exceeding the environmental tolerances of the site. In particular, a primary objective of canal design is to avoid exceeding the capacity of the waterway and the receiving waterbody to assimilate pollutants.

8.2 Canal Design Criteria

A canal design *criterion* is a qualitative statement establishing a definite value or range of a variable which will be used in the specific design.

Table 8.1 - An Example of Design Objectives, Guidelines, Criteria, and Constraints for a Hypothetical Project.

Examples of Canal Design Objectives

Type of Community: 1/4-Acre Lot, $20,000; home $50,000.
Source of Water Supply: offsite.
Method of Handling Waste Water: retention basins; septic tanks where feasible; otherwise offsite.
Extent and Navigability of Canals:
- one major loop with several branches,
- large boats limited to principal canal and marina
- small hand-powered boats in all canals

Development is to handle drainage from 25-yr, 6-hr storm.
Magnitude of Investment: $500,000
Time Span for Total Work: 2 yrs after receipt of all permits.

Examples of Canal Design Guidelines

Canals to follow natural contours wherever feasible.
Septic tanks to be used where soil conditions are suitable.
Banks of canals to be naturally-vegetated slopes.
All drainage except on canal bank to be directed, by means of swales, to detention ponds.

Examples of Canal Design Criteria

Flushing of normal pollutant load to be less than 20 hr under normal climatic conditions.
Canal bank slopes to be 1:5.
Canal depth to be between 4 and 8 ft.
Canal surface width to be between 80 and 100 ft.
Only one tidal entrance at specified location.
Class III water characteristics to be maintained.
Open area/developed area ratio to be ≥ 0.25.
Orientation of axis of poorly flushing channel to be within $\pm$ 10 degrees of prevailing summer wind direction.

Examples of Canal Design Constraints

Development Constraints
requirement for preservation of a natural area
only one tidal entrance feasible due to limited shoreline
Legislative Constraints
Federal, State, regional and local
Technical Constraints
limitation on degree of flushing by natural forces

Table 8.2 - Some Examples of Principal Design Objectives Relating to Residential Canal Design.

Objectives and Considerations	Range of Decisions
1. Define investment parameters	
level of investment	equity required
duration of investment	loan restrictions
time span of total work	1. obtaining permits 2. after permits
2. Define type of community	
a) potential market characteristics	1. Lot size: large, small, or mix 2. Lot density 3. Lot cost
b) character (appearance) of the community and its aesthetic appeal	1. Urban vs. rural 2. Developed area/open area ratio: a) areas reserved b) degree of preservation of shoreline
c) degree of access to water	1. Access from: a) each lot b) marina only 2. Both individual docks and marina
d) navigability of canals and basin(s)	1. Tidal entrance(s): a) number b) locations 2. Canal depths and widths desired 3. Marina depth and shape desired
3. Source of water supply	1. Ground water (onsite) 2. Offsite supply
4. Method for handling waste water	
a) sewage (sanitary)	1. Onsite a) septic tanks b) central treatment 2. Offsite
b) sewage (storm)	1. Direct storm sewer 2. Detention/Retention 3. Curb and gutter or swales

Most design criteria will follow directly from the design objectives, but they will be guided or restrained by design guidelines and design constraints. For example, the septic tank guidelines described in Chapter 7 might be used by the designer to establish the (hypothetical) criterion that "septic tank absorption fields will be located 200 feet from the mean high water line in any canal."

The design *criteria* will be developed in detail by the iterative design process. A trial canal design will be established initially to be compatible with the overall plan for the development, using topographic maps, aerial photographs, and additional archived data as well as data provided by preliminary site surveys. Some of these criteria will interact with others, so the designer may prefer to specify a range for one variable, as a limit within which he will optimize other more important variables. Thus, for example, rather than specifying the depths of all channels in a canal network to be 5 feet, he may attempt to find the depth between 4 and 8 feet which maximizes the flushing rate of the canal. This will establish more preliminary criteria defining canal widths and depths, side slopes, the possible location of a boat basin, the drainage plan, and other details which will be used to define a canal network for model tests. Then, as these tests proceed, the criteria may have to be modified in order to achieve acceptable circulation and flushing in the final design.

8.3 Canal Design Guidelines

A design guideline is a qualitative statement or set of statements that can assist the designer toward meeting his design objectives. For example, a design guideline related to residential canals could be:

> "Natural methods of erosion protection, such as the planting of wetland vegetation, should be used on canal banks wherever feasible."

Design guidelines have been written for a variety of different kinds of projects by planners, federal, state and local regulatory agencies, construction industry associations, and consulting engineers. Guidelines are nearly always written from a particular viewpoint: for example, one author may be committed to preservation of the environment, while another may be primarily interested in achieving the least cost in construction. Guidelines also are based on the experiences of the author, and may, therefore, be incomplete or perhaps overly biased.

In order to compile a unified set of canal design guidelines, individual sets of guidelines have been obtained from six different sources and have been combined with other concepts developed by the Hydraulic Laboratory during the Canal Flushing study. The six original sources included all of the major decision-making areas involved in coastal development: federal government, state government, regional planners, the construction industry, conservation, and engineering. Each set of these guidelines is included in Morris, Walton & Christensen [1978, pp. 447-477].

The unified set of guidelines developed for canal and canal network design is included in Table 8.3 at the end of this chapter. It is oriented primarily toward hydrodynamic considerations, or factors which directly affect the hydrodynamic aspects of design. The guidelines are subdivided in accordance with identifiable design elements, such as canal banks and beds. Within each design element category, specific guidelines are identified. This arrangement will provide the canal designer with a convenient reference for making design decisions.

8.4 Canal Design Constraints

Canal design constraints are the limits imposed by the site itself, natural forces at the site, the availability of engineering and construction expertise, economics, and the regulatory process. Design constraints can normally be expressed quantitatively: number of acres available, mean tidal range, costs of construction, and permissible levels of dissolved oxygen deficit, for example. The design elements relating to the geometry and operating characteristics of a canal network are described beginning in Chapter 11, while some of the other considerations, particularly the legislative criteria, are presented in the remainder of this chapter.

8.4.1 Development considerations. The developer's fundamental objective is to balance the opportunities for development which are inherent in the site and the regulatory restrictions to that development, to maintain an adequate cash flow and to realize a profit that will justify his investment. Every aspect of the process can be viewed in terms of cost, although it is not always easy, or even possible, to assign realistic costs to some components of the problem. In particular, the preservation of natural systems, the enhancement of natural site characteristics, and protection of the development from wind and storm damage all represent an economic cost. The preservation of a natural area at the site represents a loss in the number of units which can be offered for sale, but may increase the value of the remaining units if the vegetation serves to control drainage or represents an additional aesthetic attraction to the buyer. Likewise, the cost of protection to tidal entrances, canal banks and dwellings against storm damage is reflected in the increased cost of construction materials and establishing vegetation, but represents a saving to the developer if proper construction and canal bed and bank stabilization avoid storm damage and law suits based on inadequate construction. Some of these costs are legislated into design through local, state and federal regulations. This is necessary to avoid needless waste or destruction of natural resources, incompatible neighboring land uses, and other major long-range problems. At the same time, too much detailed regulation can stifle creative design.

Time and effort expended in planning, and determination to do a thorough job of design, will be paid back through fewer problems and greater profit in the long run. There are many sources of free advice which are applicable and valuable, such as detailed outlines of the development process and environmental checklists [for example, see Morris, Walton & Christensen, 1978, pp. 421-425]. Local planning and government agencies are available to explain regulations and to assist in design decisions, since they would prefer that a design be initially well thought-out to minimize future problems and adverse impacts.

8.4.2 Legislative constraints. All four levels of government, -- federal, state, regional and local -- have an interest in the protection and utilization of important natural resources. The coastal zone of the United States is recognized by many citizens as being one of these resources. During the past decade a national concern over damage to parts of the coastal environment has led to an increasingly complex and far-reaching response from the federal and state governments, in terms of legislative acts, regulations, and planning incentives.

A problem arose in the late 1960's when it became apparent that certain coastal developments were causing a broad spectrum of environmental problems, including irreparable damage to valuable natural resources. Examples of the loss of wetlands by dredge and fill for canal development are but one of a variety of activities which did not, in their economic justification, take into account the true cost to the state and to the nation of the loss of some of its most valuable natural areas. The federal government, responding to an expression of concern throughout the country, at first developed controls over the most obvious violations of environmental common sense: wetlands dredge and fill, the dumping of pollutants in the nation's waters, and the destruction of intertidal land. After this initial response, it began to evolve (and continues to evolve) a philosophy of control which encourages the development of state and local capabilities to rationally manage the resources of the coastal zone according to local needs.

a) Federal Authority

The authority exercised by the federal government over the activities associated with the construction and maintenance of residential canal systems in the coastal zone is based in part on an awareness of the need to restrict certain of these activities in the national interest. In particular, the intent of the federal government is to carefully control all future development in wetlands, and the burden of proof is on the applicant to demonstrate that no *practical* alternatives exist and that no *unacceptable* adverse effects on the wetlands will occur. This policy has been kept intact in state and local regulations, and applies to "all activities involving the discharge of dredged or fill material in *navigable waters*" [EPA, 1975b, p. 41292]. In 1977 the Corps of Engineers redefined *navigable waters* (inasmuch as they apply to residential canals in the coastal zone) as "navigable waters of the United States":

> *Navigable waters of the United States* means those waters of the United States that are subject to the ebb and flow of the tide shoreward to the mean high water mark and/*or* are presently used, or have been used in the past, or may be susceptible to use to transport interstate or foreign commerce... Man-made nontidal drainage and irrigation ditches excavated on dry land that feed into navigable waters will not be considered "waters of the United States" under this definition. (Emphasis added). [Corps of Engineers, 1977, p. 37144]

The regulations regarding filling of wetlands do not generally apply to upland canals, but filling of wetlands is only one of the coastal activities which is regulated by the federal government. The authority for regulation of all activities in or affecting *navigable waters of the United States* is vested in the Department of the Army, through the Corps of Engineers.

This authority has been recognized since the enactment of the River and Harbor Act of 1899, commonly called "The Refuse Act", which was adopted to "protect navigation and the navigable capacity of the Nation's waters" [Corps of Engineers, 1977, p. 37122]. Originally, the administration of this act applied only to navigation. In December, 1968, the Department of the Army revised its policy with respect to the review of permit applications and published a "list of additional factors besides navigation that would be considered in the review of these applications". These included: "fish and wildlife; conservation; pollution; aesthetics, ecology; and the general public interest," [Corps of Engineers, 1977, p. 37122] which was upheld by judicial test.

Beginning in 1969 with the passage of the National Environmental Policy Act (PL 91-190), a series of laws to clarify jurisdictions, definitions and the objectives of planning activities, were issued [Morris, Walton & Christensen, 1978, p. 427]. The principal objective of these extensions and clarifications was to provide specific protection to important resources from degradation and pollutants in addition to the products of dredging. One of these acts is the Federal Water Pollution Control Act (PL 92-500) enacted in October, 1972, with the announced purpose of "restoring and maintaining the chemical, physical and biological integrity of the Nation's waters" [Corps of Engineers, 1977, p. 37123].

Another of these acts, which is most important from the viewpoint of state and local authority, is the Coastal Zone Management Act (CZMA) of 1972 (PL 92-583). This legislation places the responsibility for planning the management of the coastal zone upon the coastal states, on a totally voluntary basis.

b) State Authority

As a result of relatively recent changes in the organization of the government of the State of Florida, the primary responsibility for regulation of development in the coastal zone lies with the Department of Environmental Regulation (DER) and the Department of Natural Resources (DNR). In addition, the state is developing a comprehensive Coastal Management Program (CMP) designed to meet both state legislative requirements and the requirements of the federal CZMA.

Partially to satisfy the requirements of the federal CZMA, the state is in the process of inventorying and designating geographic areas of particular concern. There are five major programs for the management of these areas within Florida's coastal zone [Bureau Coastal Zone Planning, 1977, p. 30].

1. Aquatic Preserves Program

2. State Wilderness System Program

3. Environmentally Endangered Lands Program

4. Areas of Critical State Concern (ACSC) Program

5. Coastal Construction Setback Line Program.

The owner and canal designer should be aware of the locations of the designated areas within these programs, as well as the locations of areas being considered for this program, and realize that developments within or in close proximity to these areas will be much more difficult to permit than in other locations.

A special flood hazard area is defined as an area which has a 1 percent annual chance of flooding (or, is subject to the hundred-year flood). However, the size of the design storm for which a development must be designed may or may not be defined by local or regional planning ordinances. For overall guidance there is the Federal Flood Insurance Program, administered by the Department of Housing and Urban Development (HUD) through the Flood Disaster Protection Act of 1973. The act requires HUD to identify and notify communities having a flood hazard area, and the community must then either make prompt application for participation in the federal flood insurance program or must satisfy the secretary of HUD that the area is no longer flood prone [Clark, 1977, p. 790].

c) State Legislation

The specific state legislation applicable to canal projects and related construction work is summarized in Table 8.4, taken from a more extensive table in the CMP workshop draft [Bureau Coastal Zone Planning, pp. 66-67]. For projects not in the Development of Regional Impact (DRI) category, six state agencies are involved, of which two, the DER and the DNR, have the primary responsibility.

DER will evaluate the potential impact of the proposed project on the waters of the state primarily in accordance with the provisions of Chapter 253, Florida Statutes (FS), or Chapter 403, FS. Chapter 253, Land Acquisition Trust Fund (commonly called the "State Lands Act"), establishes restrictions on filling land and dredging (Sect. 253.123). Under this legislation DER will determine if the project will cause "harmful obstruction to or alteration of the natural flow of navigable waters; will induce harmful or increased erosion, shoaling of channels or create stagnant areas of water; will interfere with the conservation of fish, marine and wildlife or other natural resources; will induce destruction of ... marine productivity including ... natural marine habitats, grass flats..., marine soils..." [Corps of Engineers, et. al., 1977, p. 5]. Chapter 403 FS, Part I Pollution Control, establishes restrictions on disposal of waste water and sewage. Under this legislation DER will "determine if the proposed project will degrade the quality of the water by destruction of resources which maintain water quality or will degrade the quality of water by discharging materials harmful to the environment" [Corps of Engineers, et. al., 1977, pp. 5-6].

DNR will evaluate the functionality of the proposed construction and its compatibility with existing coastal processes at the location for the construction in accordance with the provisions of Chapter 161, FS, the Beach and Shore Preservation Act. "An evaluation will be made of the protection afforded against coastal flooding and storm induced erosion and of the physical impact on adjacent properties" [Corps of Engineers, et. al., 1977, p. 6]. All activity undertaken specifically for shore protection must have a permit, as well as all other structures and physical activity which by their nature and design might have similar effects, including breakwaters, seawalls and artificial nourishment or other deposition or removal of beach materials. Docks and similar structures are also included if primarily of a solid or highly impermeable design.

Table 8.4 - Authorities to be Utilized and State Agencies Involved in Activities Related to Canal Development.

Uses/ Activities Subject to Management	State Authorities to be Utilized: Ch. 403 F.S.	Ch. 381 & 387 F.S.	Ch. 266 F.S.	Ch. 161 F.S.	Ch. 177 F.S.	Ch. 253 F.S.	Ch. 373 F.S.	Ch. 258 F.S.	Ch. 372 F.S.	Ch. 267 F.S.	Ch. 23 F.S.	Laws of FL 77-306	Ch. 216. 212 F.S.	Laws of FL 77-375	Ch. 193 F.S.	Ch. 418 F.S.	Ch. 592 F.S.	Ch. 171 F.S.	Ch. 403. 165 F.S.
Private Docks	DER					DER DNR		DNR											
Dredging and/or Filling	DER			DNR	DNR	DER DNR	WMD	DNR	G+F	DOS		DER	DSP	G+F				DNR	DER
Sewage Treatment/ Disposal	DER	HRS					WMD	DNR	G+F		DSP	DER	DSP		DOR DER				
Discharges into State Waters	DER					DER DNR	WMD	DNR											DER
Septic Tanks		HRS																	
Water Wells	DER	HRS					WMD												
Shoreline Erosion Control Structures	DER			DNR	DNR	DER DNR												DNR	
Beachfront Development	DER			DNR															
Historic Preservation			DOS							DOS						DNR	DNR		

State Authorities:

Ch. 403 F.S.: Air and Water Pollution Control
Ch. 381 & 387 F.S.: Sewage Treatment and Disposal
Ch. 266 F.S.: Historic Preservation
Ch. 161 F.S.: Beach & Shore Preservation Act
Ch. 253 F.S.: State Lands
Ch. 373 F.S.: Water Resources Act
Ch. 258 F.S.: Aquatic Preserve Act
Ch. 372 F.S.: Game & Freshwater Fish
Ch. 267 F.S.: Archives & History Act
Ch. 23 F.S.: State Comprehensive Planning Act/Clearinghouse
Laws of FL 77-306: Coastal Zone Planning & Management
Ch. 216. 212 F.S.: Clearinghouse (Federal A-95)
Laws of FL 77-375: Endangered & Threatened Species Act
Ch. 193 F.S.: Tax Assessments
Ch. 418 F.S.: State/Local Recreation Coordination
Ch. 592 F.S.: Recreation & Parks
Ch. 177 F.S.: Coastal Mapping Act
Ch. 403. 165 F.S.: Pollution Recovery Trust Fund

Implementing Agencies:

DER: Dept. Environmental Regulation
DNR: Dept. of Natural Resources
HRS: Dept. Health & Rehabilitative Services
DOS: Dept. of State
G&F: Game & Fish Commission
WMD: Water Management Districts
DSP: Division of State Planning

Additional Authorities and Implementing Agencies if DRI Project:

Ch. 380 F.S.: Environmental Land & Water Management Act - DSP: Div. of State Planning
Ch. 478 F.S.: Land Sales - DBP: Div. of Business Regulation
Ch. 377 F.S.: Conservation of Oil and Gas Resources - DNR: Dept. of Natural Resources
Ch. 376 F.S.: Oil Spill Prevention - DNR: Dept. of Natural Resources
Ch. 163 F.S.: New Communities Act - DCA: Dept. of Community Affairs
Ch. 160 F.S.: Regional Planning Councils - DSP: Div. of State Planning

Additional Authorities and Implementing Agencies if Development in ACSC:

Ch. 380 F.S.: Environmental Land & Water Management Act - DSP
Ch. 163 F.S.: Local Comprehensive Planning Act - DSP
Ch. 478 F.S.: Land Sales - DBR
Ch. 377 F.S.: Conservation of Oil & Gas - DNR
Ch. 376 F.S.: Oil Spill Prevention - DNR
Ch. 160 F.S.: Regional Planning Councils - DSP

Another important regulation of which the canal designer should be aware is the classifications of the waters of Florida in terms of usage (Chapter 17-3, 403 FS, Pollution of Waters). Class I waters are public water supply; Calss II waters are designated for shellfish harvesting; Class III waters meet criteria for recreation and propagation and management of fish and wildlife; Class IV waters are classified for agriculture and industrial water supply; Class V waters are designated navigation, utility, and industrial use. The criteria for any classification of waters are readily available from the local office of DER.

d) Regional and Local Authority

At the regional and local levels control over land use and development is exercised through planning, zoning, subdivision regulations, and other means such as housing codes and the enforcement of federal minimum property standards. Land use controls, and questions about their effectiveness and limitations, are complex issues. Both the developer and the canal designer need to be aware of the regulations in force and how to use them to their advantage in the overall design of the canal system.

A checklist for the regulatory portion of the residential canal development is provided in Table 8.5. This summarizes the various regulations and permits which will apply to a typical residential canal development, and the agencies which are responsible for approving a given project.

8.4.3 Permitting Procedure

In 1977 the process of applying for a permit for work involving construction, dredging, and filling in the waters of Florida was simplified by the adoption of a joint federal/state procedure. A booklet listing the applicable federal and state legislation and the type of work requiring a permit, describing the processing of applications, and including specific instructions and forms for use in the application [Corps of Engineers, Florida DER and Florida DNR, 1977] is available from local offices of DER and DNR.

The construction of an artificially-created channel or canal used for recreational, navigational or other purposes, that is connected to navigable waters, requires a permit under federal law. Federal jurisdiction extends to work or structures in all tidal areas channelward of the mean high water line, as well as in all rivers, streams and lakes to the ordinary high water line, and in marshes and shallows subject to periodic inundation. The federal review of an application for construction in Florida is conducted by the Jacksonville District of the Corps.

Unless specifically exempted, all dredging and filling activities which are to be conducted in, or connected directly to, or via, an excavated waterbody or series of excavated waterbodies to natural waters of the state require permits. Waters owned entirely by one person other than the state are included only if there is a discharge on other property or water [Corps of Engineers, Florida DER and Florida DNR, 1977, p. 4].

Activities requiring a permit include the construction of piers; docks; mooring piling; excavation; filling; disposal of dredged material; riprap and revetments; retaining walls; beach restoration; levees; wire or cable

Table 8.5 - Checklist for Regulatory Information.

RULES AND REGULATIONS	AGENCY TO CONTACT FOR INFORMATION
( ) Background Information, Including Overview of Regulations and Requirements	Planning Commissions
( ) Zoning Regulations	Planning Commissions Building and Zoning Officials
( ) Subdivision Regulations & Registration	Planning Commissions Florida Division of Land Sales and Condominiums
( ) Construction Codes	Building Officials
( ) Permits for Septic Tanks	County Health Department Florida Department of Health and Rehabilitative Services
( ) Regulations for Private Wells	County Health Department Florida Department of Health and Rehabilitative Services Regional Water Management District
( ) Permits for Private Docks, Bulkheads, Other Structures in Waterways	Florida Department of Environmental Regulation Florida Department of Natural Resources Jacksonville District, U.S. Army Corps of Engineers
( ) Permits for Alteration of Wetlands	Florida Department of Environmental Regulation Florida Department of Natural Resources Regional Water Management District Florida Game & Freshwater Fish Commission
( ) Permits for Bridges	Florida Department of Transportation U.S. Department of Housing and Urban Development
( ) Flood Plain Regulations	U.S. Department of Housing and Urban Development
( ) Environmental Protection Regulations	Florida Department of Environmental Regulation Florida Department of Natural Resources
( ) Historical and Archaeological Sites	Florida Department of State Florida Department of Natural Resources

over the water; pipe, wire or cable under the water; clearing; channel and upland canal construction; intake and outfall pipes and/or structures; and the transportation and deposition of dredged material for open water dumping. Permits are required from the U.S. Coast Guard and the Florida DER for bridges and overhead pipelines. Permits for discharges of other than dredged or fill material must be obtained from the appropriate water pollution control agency listed in the permit application booklet [Corps of Engineers, Florida DER, and Florida DNR, 1977, pp. 4, c-1 and c-2].

Projects which are exempted from DER permitting procedures include maintenance dredging of existing man-made channels, and intake and discharge structures for ten years from issuance of the original permit granted prior to July 1, 1975, under the conditions specified in the permit application booklet. Also the installation and repair of mooring pilings associated with private docks, construction or restoration of seawalls, maintenance of drainage ditches, repair or replacement of existing pipes from stormwater runoff, construction and maintenance of swales, and other specific activities, are exempted under certain specific conditions [Corps of Engineers, et. al., 1977, Appendix B].

If a project is large enough to fall under the jurisdication of the Flordia Environmental Land and Water Management Act (Chapter 380, FS), wherein certain defined activities having impact on more than one county are subject to regional and state review, then the project is classified as a Development of Regional Impact (DRI). If the development is to be located in a designated area of critical state concern (ACSC), or within the one-hundred year hurricane flood zone, it will also require special management consideration from the state [Bureau Coastal Zone Planning, 1977, p. 4]. If the Corps determines that granting the permit would constitute a major federal action and that the proposed project would have a significant effect on the human environment, an Environmental Impact Statement (EIS) will be prepared prior to final action on the permit, as required by the National Environmental Policy Act of 1969. The Corps will prepare the EIS, but the applicant will be required to submit the necessary data. When the state requires a DRI, the Corps will, if not legally constrained, use the DRI application to prepare an environmental assessment to aid in avoiding the delays inherent in preparation of an EIS [Corps of Engineers, et. al., 1977, p. 5].

Table 8.3 - Canal Design Guidelines -- Hydrodynamics

Design Element	Specific Characteristics	Guideline
Banks	Stablization	Sloping, vegetated banks should be used along as much of the canal as practical to provide protection against erosion.
	Slope	Slope should be determined for protection against the highest practical design flow. Multiple slopes are one possible alternative. Geotechnical slope stability must be assured. Piping to be avoided.
	Protection	Vegetation provides most economic bank protection.
	Aquatic life	Sloping banks provide habitat for shallow-water fish and wildlife.
	Curvature	Curvature of banks induces secondary flows which assist in vertical mixing.
	Discontinuities	Banks should transition gradually from slopes to bulkheads.
Beds	Holes	Beds should not have discontinuities such as dredge holes or sills. Deep depressions act as nutrient traps and do not easily flush.
	Stabilization	Beds can be protected against erosion, in some instances, by introducing aquatic vegetation. Deposition can be controlled to some degree by smoothing discontinuities in the geometry that can cause local low velocities, or reducing erosion in other parts of the canal network.
	Roughness	Bed roughness may be artificially increased to increase vertical mixing, which utilizes more of the total energy in the flow and may thereby reduce flushing in other parts of the network.
Boats Basins and Marinas	Flushing	Basins should be designed with a smooth entrance transition, same depth as canals, and favorable circulation for flushing. Proper orientation with prevailing wind substantially assists flushing.

	Access channels	Basin access channels should be designed to maximize flushing and avoid stagnant or recirculating areas.
	Existing shoreline configurations	Basins should be planned to minimize the extent of excavation, shoreline alteration, and disturbance of vital habitat areas.
	Waste handling	Marina design must incorporate facilities for proper handling of sewage, refuse, and waste.
	Siting	Basins should be sited to avoid boat traffic across shallow grass flats or in locations where boat wakes could be harmful to vegetation or unstabilized banks.
Bulkheads	Usage	Use of vertical bulkheads should be minimized in canal networks.
	Siting	Bulkheads, if used, should always be located shoreward of wetlands.
	Permeability	Bulkheads are often collapsed by infiltration and erosion onthe land side. Proper drainage, or a permeable structure, and footing extending far enough below the bed need to be considered.
	Reflection of wakes	The adverse effects of reflected boat wakes may be minimized by alternating bulkheads on both sides of a canal (facing each bulkhead with a vegetated sloping bank directly across the canal).
Circulation	Overall in network	Circulation throughout the canal network is enhanced by smooth transitions and increased tidal prism at inland extremity of canal network.
	Vertical circulation	Vertical motion of water is caused by turbulence,wind, bends, salinity gradients, and large rougness elements.
Curvature	Canals	Curvature of canals induces helical secondary flow which enhances vertical mixing and persists some distance downstream of the bend.
	Aesthetics	Curved channels can provide a more natural appearnce, especially if banks are sloped and vegetated.

Dead-ends	Circulation	Dead-end reaches have substantially less circulation than flow-through canals.
	Wind	Dead-end reaches must be aligned with prevailing wind to provide an opportunity for intermittent flushing. Orientation should be with wind away from dead-end. Opposite orientation permits wind driven waters to carry debris into canals, which can settle to bottom, decay, and increase local BOD.
Depth	Photic limit	Excessive depth precludes light penetration to the bed, where aquatic vegetation can grow under proper conditions.
	Transitions	Depth of canal should be the same as, or gradually transition from, other adjacent sections in the canal system.
	Geometry	Depths, widths, and other parameters need to be determined by comparative simulations using a numerical model. Depths cannot be determined independently.
Detention	Design	Detention reservoirs should be designed for a specific design storm and drainage plan.
	Control structures	Detention systems permit controlled amounts of infiltration and release at a flow approximating the predevelopment volume.
Drainage	Saltwater intrusion	Drainage of wetlands decreases freshwater storage capacity, both at the surface and in the aquifer, which may permit further intrusion of the saltwater interface.
	Patterns	Before development occurs, natural drainage patterns of the site and the surrounding region should be identified and incorporated into the site design. This will avoid interrupted flows and will enable the new design to work with established drainage patterns, which should not be modified any more than necessary.
	Wetlands	Wetland areas should not be drained or developed. These are too valuable in their natural conditions as nurseries for aquatic life.

Dredging	Canal beds	Hydraulic dredging can result in uneven canal bottoms, and therefore must be used with skill and caution.
	Wetlands	No dredging should be permitted in productive wetlands.
	Aquifer	Dredging into a shallow freshwater aquifer will accelerate the loss of freshwater to the sea and the intrusion of the saline interface farther into the aquifer.
	Canal entrances	Canal entrances should be located so as to minimize destruction of fish and wildlife habitat. An entrance must be sited so that the channel, and boat traffic, will traverse the smallest amount of productive wetland vegetation or shallow productive bottom substrate.
	Navigation channel size	Navigation channel dimensions should be kept to the minimum size consistent with circulation requirements in the system.
	Dredging operations	Dredging operations should be stopped during critical periods of fish migration and feeding. If the operation interfers in such migration and feeding.
	Dredge types	Dredge types should be selected so as to minimize environmental disturbances during operation.
	Dredge spoil	Dredge spoil should not be disposed of in open estuarine or receiving waters or in vital areas. Curtains should always be used.
Entrance Design	Location	Canal entrances should be located to minimize destruction of fish and wildlife habitat during construction.
	Minimizing erosion	Entrances and associated offshore channels should be situated and marked so as to minimize erosion of adjacent or opposite shoreline by boat traffic.
	Gradual transition	Entrances should have a gradual hydraulic transition from the interconnecting waterbody.
	Stabilization	Entrance banks should be sloped for shoreline stabilization.

Erosion	Stable cross-sections	Beds and banks should be designed with stable cross-sections to prevent erosion.
	Vegetation	Beds and banks should be stabilized with vegetation to prevent erosion.
	Construction details	Bulkheads should be properly designed with filter cloth, graduated fill, tie-backs, wingwalls, adequate toe penetration or other firm foundation, and adequate drainage to prevent erosion.
Fill	Wetlands	Fill should not be placed in wetlands.
	Foundations	If fill is to be used for foundations, it must be properly graded and compacted with minimal destruction to surrounding vegetation.
	Raised foundations	Raised foundations (piles) will eliminate the requirement for additional fill, if this technique is acceptable to permitting agencies.
	Dredging	Fill should not be dredged indiscriminately from canals, and preferably should be obtained from offsite. The canal designer must provide appropriate instructions to the contractor regarding the depth and uneveness of dredging that will be permitted.
	Drainage	Fill must be placed and prepared with adequate drainage.
Flushing	Transitions	Design channels with gradual transitions for lowest energy loss.
	Tides	Small amplitude tides have limited effectiveness in flushing.
	Length of canals	Canals extending long distances from canal entrances will require additional energy sources (such as moderate or high prevailing winds) to provide adequate flushing.
	Wind effect	Flushing effectiveness of winds is maximized by orientation of channels in direction of prevailing winds.
	Bends	Channel bends increase flushing through induced vertical circulation.

	Roughness	Local flushing may be increased by increasing the size of roughness elements in the channel. The use of roughness must be balanced with the available energy budget.
Groundwater	Offsite supply	Consider off-site supply to minimize groundwater withdrawal.
	Drainage plan	Maximize groundwater supplies by an appropriate drainage plan.
	Saltwater intrusion	Limit groundwater withdrawal to minimize saltwater intrusion into the aquifer.
	Subsidence	Limit groundwater withdrawal to avoid subsidence of land.
	Absorption fields	Design septic tank absorption fields for adequate filtering before seepage into groundwater supplies and canals.
Mangroves	Preservation	Mangrove areas bordering coastal waters or tidal channels should be preserved as buffer zones to maintain water quality and shoreline protection. Mangroves also help to retard runoff, maintain aesthetic values, and maintain habitat for aquatic life and birds.
	Permitting	Mangroves should be preserved to minimize permitting problems.
	Re-establishment	Mangroves should be re-established in shallow areas suitable for their propagation.
Mixing	Roughness	Use roughness elements to increase local turbulent mixing.
	Bends	Channel bends can be used to produce local vertical mixing through the action of helical flows.
	Wind	Use channel alignment with the prevailing wind to increase vertical mixing.
Natural Preserves	Protection and Aesthetics	Allocate a significant portion or portions of the site for natural preserves to protect wildlife and retain aesthetic appeal.

Recharge	Drainage	Design drainage plan to retain all runoff for recharge to the aquifer.
	Retention and Detention	Direct-runoff to vegetated swales and retention or detention ponds rather than to curbs, gutters, and storm drains.
	Paved areas	Minimize paved areas and roof area to maximize area available for recharge.
	Wells	Avoid the use of deep wells into the Floridian Aquifer unless the wells are properly cased and sealed. Otherwise deep wells may contaminate the shallow freshwater aquifer.
	Criteria	Use the following criteria for delineating an area suitable for aquifer recharge: 1. Geologic formation must be permeable. 2. Water must be available in the form of rainfall or runoff. 3. Maximize area of permeable surface. 4. Provide recharge area sufficient to balance depletion of water through evaporation, runoff into canals (if any), and usage. 5. Recharge area should be relatively pristine. Agricultural lands and areas near sewage outfalls are least desirable.
Retention	Runoff	Provide storage areas to retain runoff which cannot be handled by detention.
	Volume	Retention volume should be calculated by considering a suitable design storm and the drainage plan for the site.
Revegetation	Preservation	Minimize requirements for revegetation by preserving as much vegetation as possible during construction.
	Construction	Revegetate areas needing vegetation for runoff control as soon as possible after construction.
	Indigenous species	Revegetate shallows and banks with appropriate plant species.
	Plant communities	Provide for plant communities that are mutually compatible.

Roughness	Distribution	Distribute bed and bank roughness as required in the canal network to avoid stagnant areas.
	Energy losses	Control unwanted energy losses by minimizing local roughness as necessary.
Runoff	Control	Control site drainage by means of vegetated swales, filter mounds, retention or detention ponds, and vegetation to preclude runoff into canals.
	Erosion	Control drainage so that runoff cannot cause erosion.
	Septic drain fields	Control drainage so that runoff cannot affect operation of septic absorption fields.
	Separation	Separate stormwater drainage from waste water disposal systems.
Saltwater Intrusion	Groundwater table	Maximize the potentiometric head (elevation of the groundwater table) on. and in the vicinity of, the site to minimize saltwater intrusion.
	Drainage	Retain as much drainage water as possible on site to minimize saltwater intrusion.
	Off-site supplies	Use off-site water supplies to minimize effects of well drawdown.
	Hydrologist	Use the services of a competent hydrologist to obtain measurements on site if saltwater intrusion is a problem.
Sedimentation	Water velocity	Design canals for adequate velocities to carry suspended sediments out of canals.
	Banks and beds	Design canals with stable banks and beds to minimize the amount of eroded material in suspension.
	Sediment control: · canals	During construction, use sediment blankets or barriers in the canals to avoid sedimentation of offshore areas.

	• land	During construction use preventive measures, such as sediment basins, to keep sedimentation from runoff out of canals, and after construction promptly replant cleared areas.
Septic Tanks and Drain Fields	Alternatives	Consider alternatives, such as offsite waste treatment or onsite package treatment plants, as substitutes for septic tank systems.
	Location	Do not install septic tanks or drain fields close enough to canals to permit leaching into the canals or high water flooding of drain fields.
	Tests	Test borings should be conducted prior to construction to determine soil types and hydrologic conditions.
	Soils	"Septic tank systems should be installed only when soil characteristics are suitable" [Clark, 1977, p. 505].
	Setback	"The absorption field of a septic tank system should be set back at least 150 feet from the annual high water line" [Clark, 1977, p. 503].
	Groundwater level	"Septic tank systems should be installed only when the highest annual groundwater level is at least four feet below the absorption field" [Clark, 1977, p. 504].
Shallow areas	Vegetation	Include large shallow areas in the canal system where practical to provide space for re-establishment of intertidal vegetation, which provides nutrient uptake and a varied habitat for aquatic animal species.
Sills	Plug	Do not permit a sill to remain at the mouth of a canal after a plug has been removed.
	Communicating waters	Do not dredge canals deeper than the depth of the communicating waters. If this is necessary, provide a gradual transition from the entrance into the canal.

Site Characteristics	Before purchase	Determine hydrology of site and surrounding area, and hydraulic characteristics in the communicating waterbody, before purchase of the site.
	Before construction	Determine site characteristics on both long and short term bases from available information and extensive measurements before construction.
Slopes, Bank	Stability shallows wind	Use gently sloping banks to provide channel stability, intertidal shallows and greater canal surface area for wind-induced flow and for oxygen transfer from the surface.
Soils	Before purchase	Determine characteristics of soils at the site before purchase.
	Bearing capacity	Check soil bearing capacity prior to moving heavy equipment on to the site.
	After construction	Make certain that all soils that can sustain plant life are revegetated as soon as possible after construction.
Stabilization	Methods	Stabilization is best provided by vegetated sloping banks, then rip-rap on a sloping bank, and least, by bulkheading.
	Inlet	Provide a smooth transition from the communicating waterbody into the canal and integrate it with a mild sloping bank while providing the required protection against storms.
	Control erosion	Control erosion during site preparation and construction by planting buffer strips of natural vegetation, and providing artificial detention systems and runoff diversions.
Tides	Differential	Use more than one tidal entrance especially if they will be sufficiently separate to provide an elevation difference.
	Neap tide	Base design calculations on neap tide amplitude which provides the lowest amount of tidal energy to a canal system.
	Tidal prism	Use tidal prism as a design tool to increase flow through interconnecting channels and basins such as marinas.

Vegetation	During construction	Do not destroy or remove any more native vegetation than necessary during construction.
	Native varieties	Plant native, indigenous vegetation in preference to exotic species.
	Stabilization	Use vegetation to stabilize beds and banks, control runoff, provide habitat and aesthetic appearance, take up nutrients, and provide screen between dwellings.
Waste Disposal	Control	All waste waters should be controlled until they have been either treated, or disposed of offsite in an environmentally safe way.
	Mixing	Waste waters and storm waters should not be mixed, since only waste waters require treatment before recycling.
	Aquifer, septic tank systems, package plants	Improper disposal of solid and liquid waste can contaminate the aquifer. Septic tanks normally are not suitable for coastal development. Package treatment plants may be considered and must be placed above the 100-year flood level.
	Marinas	Marina designs must incorporate facilities for the proper handling of sewage and waste.
Winds	Alignment	Channels with poor flushing characteristics should be aligned in the direction of the prevailing wind.
	Water surface width	The effectiveness of wind in developing vertical secondary mixing circulation is increased by increasing the width of the water surface in the canals.
	Trees	Since trees along canal banks retard the lower layers of wind, which decreases the mixing effect, their screening effect should be reduced if there is no other way to provide mixing in an existing channel.
	Simulations	In testing different canal configurations it is important to use typical variable winds for the site during simulations.

CHAPTER 9
SITE CHARACTERISTICS, AVAILABLE INFORMATION, PRELIMINARY SITE INVESTIGATIONS AND FIELD SURVEYS

The design of a canal system for a particular site should begin with an evaluation of the site characteristics. A potential site for a canal development will have a variety of canal-related characteristics which are initially unknown and unquantified. The overall objective of the initial site evaluation is to observe and evaluate the characteristics of, and opportunities presented by, the site, as well as conditions both on and offsite which could affect the overall development. It is assumed for this discussion that the usual land study has already been conducted by the developer and that the following characteristics have already been evaluated: area of property and legal boundaries, zoning, availability of utilities, transportation, existing easements and rights-of-way, proximity of public facilities, soils and erosion potential and permit requirements.

The canal-related characteristics of a site are summarized in Table 9.1.

9.1. Fixed Characteristics

9.1.1 Topography and drainage. The overall topography of a site, which includes its shoreline as well as the large-scale rises and depressions, is the principal determinant in the drainage of the site and limits the location of the canal network. Flat areas and depressions are more susceptible to flooding and are not normally suitable for housing, but are the logical choice for the construction of canals. The high areas have the driest soil, and usually provide better views. Some variation in topography is necessary for handling storm water and for protecting homesites from standing water. If the site is flat, some changes in topography may be required to control drainage.

As part of the preliminary site survey, a working topographic map for the canal design should be drawn up by the canal designer. Information drawn on this map should include at least:

- property boundaries
- vertical contour intervals of 5 ft
- major physical features such as existing canals, streams, unusual trees or vegetation, paths or roads, depressions, etc.
- extent and type of vegetation.

Table 9.1 - A Checklist of Principal Site Characteristics Relative to Canal Design

Categories to be considered from several viewpoints: onsite conditions and their effect on neighboring land; offsite conditions and their effect on the site including documentation of existing problems; and planned future offsite development.

General Components	Observable or Measurable Specific Components
Area and Boundaries	Access Points Tidal Range Bathymetry of Receiving Waterbody
Topography	Slopes Existing Waterbodies Drainage
Water Budget	Rainfall Surface Water Groundwater Infiltration Runoff Piezometric Head
Soils	Type Suitability for Construction Suitability for Vegetation
Climate	Wind Rainfall Storm Tide
Vegetation	Function Served Vital or Not Vital
Water Quality	Receiving Waterbody Existing Channels Pollution Sources
Aquatic Life	Variety Condition
Bottoms of Waterbodies	Biological Condition Chemical Condition
Aesthetics	Views Natural Features Preservation Areas

9.1.2 Tidal range. The range of the tide at a site can be estimated from the range published for the nearest subordinate station listed in the National Ocean Survey (NOS) *Tide Tables*, but this method should be used with caution since the tide can be either attenuated or amplified, or shifted in phase, in shallow coastal waters and areas with complex shorelines, and by the passage of storms. A record of the tides over a two-week period at the site, during calm weather conditions, could be correlated with the predicted tidal heights for the same period of time to obtain a reasonably accurate correction factor to apply to the mean and extreme ranges listed in the tide tables.

The determination of the legal boundary of a property with a tidal shoreline is indirectly related to the tidal range. If tidal boundaries are anticipated to be a possible future issue in the permitting or legal aspects of the development, it may be advisable to set up a long-term tide recording station and to obtain legal advice early in the project. Some specific sources for tidal data are listed in Section 12.3.2 of Morris, Walton & Christensen [1978].

9.1.3 Climate. Daily precipitation and three-hourly wind data are available from the National Climatic Center for selected stations in each state. Information on precipitation will be required for preparation of a runoff hydrograph during the design stage if there is significant stormwater flow on the site, or from neighboring areas. Rainfall frequency for any location in any state is provided on a series of maps originally available from the U.S. Department of Agriculture (see Section 12.3.3.2.B1 in Morris, Walton & Christensen [1978]). These maps give the rainfall in inches for a given return period and storm duration. Rainfall intensity-duration curves are also available from the Florida Department of Transportation, giving the intensity-duration relationships for various return periods for six homogeneous zones in Florida.

Wind is one of the most important forcing functions in canal hydrodynamics. Unfortunately, local winds have little correlation with winds measured at some other neighboring location For the simulation of the performance of a proposed canal network, however, the use of historical wind records is quite adequate. The proposed or trial canal designs may be tested for typical conditions during two or more seasons, and for these tests a representative set of variables may be hypothesized. To account for wind in such a framework, a typical sequence of wind data needs to be selected which is representative of the season to be simulated, and order-of-magnitude values will suffice. Some sources for wind data are listed in Section 12.3.3 in Morris, Walton & Christensen [1978].

9.1.4 Hydrology and water resources. The canal designer must make decisions that affect the supply, movement and quality of surface and groundwater on the site, and the interaction of these waters with water at neighboring sites on the boundaries of the property. He is therefore interested in locating, mapping and quantifying the amount and flow of surface water and groundwater resources.

In a preliminary site investigation, the canal designer should primarily act as an observer. He should look for existing bodies of surface water, determine whether they might lie in a flood plain, and try to see whether they have any visible connection with an existing canal or the receiving waterbody. He should try to determine, from local knowledge, whether such waterbodies are temporary or permanent, and attempt to locate any obvious drainage paths into or out of the waterbody. He should also take a surface and bottom sample, checking its pH, cloride content and DO. He should consider its scenic quality, note the surrounding types of vegetation and qualitatively evaluate its recreational value or wildlife habitat potential. It is also important to consider what the effect of construction activity on such a waterbody could be.

If there are existing canals, the appearance of the water and the banks can tell much about the health of the canals. Discolored water, the absence of fish and aquatic plants, odor, or floating trash may indicate the possibility of polluted water. Again, simple chemical tests can be performed to quantify basic water quality characteristics. Collapsing or unvegetated banks indicate that erosion has been taking place and that the canals are stabilizing to accommodate a higher flow than that for which they had been constructed. Shoaling, on the other hand, indicates lower rates of flow and a tendency to stabilize by decreasing cross-sections. Eroding banks may also reveal groundwater flow from a perched water table, if seepage of water through existing canal walls is detected. Accumulations of trash generally indicate poor alignment of channels with the wind.

The boundaries of the property should be investigated to determine whether there are any sources of water from neighboring properties which could affect the quantity or quality of water on the site. The shoreline, and the planned or existing tidal entrance(s), also provide information on physical trends and the quality of the water which may be of importance in future design decisions.

Groundwater is more difficult to assess. First, the canal designer should know how the planned community will interact with groundwater supplies. If a decision has already been made to provide water from an offsite source, and to take waste water offsite to an existing sewage system or a new treatment plant, the interaction with the groundwater will be minimized. In this case the principal concern is not to cut canals into the aquifer or below the water table. If no test wells are available, it would be advisable to employ a competent hydrologist to direct the tests needed to locate the water table and the aquifer, and to determine infiltration rates at potential detention basin sites.

If water is to be provided for the community from underground supplies, a hydrologic survey by an experienced hydrologist is essential. The carrying capacity of the aquifer in a given location must be estimated from observation wells, and an evaluation of the potential for saltwater intrusion into the aquifer should be made.

Some sources of information on water data, water supply and waste water disposal are listed in Section 12.3.4 of Morris, Walton & Christensen [1978].

9.1.5 Vegetation. One of the most important features of a site is its pattern of natural vegetation. In addition to its significant role in stabilizing soils, providing food and protection for wildlife, photosynthesis, and providing scenic quality, it serves as an indicator of the health of the ecosystem. The vegetation of an area invariably provides an indication of its soil characteristics and natural drainage conditions. Also, the vegetation changes with changes in the flow and quality of water, soil characteristics, climate, and other natural variables. Clearing and drainage have a particularly significant impact on vegetation, as these activities usually disturb plant succession and may establish favorable conditions for the invasion of nuisance plants or exotic (non-indigenous) species.

The canal designer, with the services of a botanist with local knowledge, should include a survey and mapping of the types of vegetation at the site in his preliminary site investigation. To a certain extent this can be accomplished with aerial photography, but poor penetration of the tree canopy and lack of resolution limit the detail that can be obtained. In conducting a biological survey, he should consider what basic function or functions each area of vegetation serves, what kinds of soils are indicated, and whether each particular group of vegetation is vital to the ecosystem. The possible effect of development activities, such as clearing, change in the water table, drainage, canal construction, change in wildlife habitat, and change in scenery, should be noted for consideration during the design of the canal network.

The zonation of vegetation on a shoreline is a particularly important indicator to the canal designer because it indicates the recent mean high water line and possible recent storm tide elevations. Vegetation in this area may be in a state of establishment or erosion. If it is in the process of establishing itself naturally, it is an indication that soil and water conditions are quite suitable for that species and that transplantation would probably be highly successful. Dying vegetation, on the other hand, could indicate the presence of a pollutant or a recent change in one or more of the natural characteristics of the site, such as water quality.

Some sources of information on vegetation are listed in Section 12.3.5 in Morris, Walton & Christensen [1978].

9.1.6 Soils. The canal design engineer needs to know the distribution of soil types on the site, because their widely-varying characteristics affect both drainage and construction decisions. Soils may drain easily, in which case they are suitable for septic tanks and detention ponds, but if they are dry and sandy, they may be too well drained and cause septic tank drina field leaching into groundwater or contamination of nearby shallow water wells. Some soils shrink and swell more easily than others, depending on the amount of absorbed water, which can lead to foundation damage. Other soils may compact well and have high bearing capacity, which is desirable for foundations of houses and roads, or be loose and unstable under load. Their suitability as topsoil for planting, or as construction material, are also factors in the design of a residential canal site.

When the canal designer conducts his preliminary site investigations, he should have a soils expert with him. The Soil Conservation Service (SCS) of the U.S. Department of Argiculture encourages developers and private landowners to consult with them on any major project before the layout is established. In many cases they will accompany the engineer and point out features which otherwise might not be noticed.

Some sources of information on soils are listed in Section 12.3.6 in Morris, Walton & Christensen [1978].

9.2 Alterable Characteristics

9.2.1 Drainage. The natural drainage at a site may be the basis for the final topographic design, if only minor modifications have to be made to the site. On the other hand, it is possible that extensive modifications to the topography are required and that the drainage of the area has to be completely changed. The canal designer should attempt to make use of as much of the natural landscape as he can, which will minimize the cost of the project and the difficulty in obtaining permits. However, he should not feel limited by existing conditions; imaginative approaches within the limits of rational engineering design can result in a substantial improvement over existing conditions at a site.

Natural drainage can only be properly assessed by observations at ground level. It is affected not only by the topography, but by the soil types, vegetation, natural swales and channels, and the amount of precipitation and evapotranspiration at the site. Historical data may be of value in determining the adequacy of natural drainage, which may not be optimal when the potential for improvement in overall site characteristics is considered.

During the preliminary site investigation, the canal designer must be alert for opportunities to improve various characteristics of the site. Particularly evident will be areas that do not have adequate drainage, such as depressions with highly impermeable soils that can flood and run off into the receiving waters or existing channels. The designer's overall viewpoint should be to preserve high ground for housing and to commit low ground for channels, and to consider whether the natural drainage is satisfactory.

Some sources of information on drainage are listed in Section 12.3.7 in Morris, Walton & Christensen [1978].

9.2.2 Pollution sources. Potential sources of pollution (see Chapter 7) are highly site-specific. It is the responsibility of the canal designer to take the necessary steps to locate and identify as many of the existing sources as possible so that they can be cited in permit applications, and so that they can either be eliminated or accommodated in the design.

Local and state governmental agencies should be cooperative in providing whatever information they have available on point sources of water pollution. This information should include the overall flow rate as well as constituent identification and concentrations.

Air pollution is also a factor in the development of a new residential canal community since it will be an area in which automotive and boat traffic will substantially increase. The canal designer should, on his site surveys, note whether there is already a large concentration of traffic, industry, or other source of air pollution nearby and take steps to make certain that his development will not aggravate existing conditions to the point where air quality standards cannot be met.

Pollution sources originating on the proposed site must not be allowed to adversely affect neighboring communities. As the canal design proceeds, the designer should check at the site as often as is necessary to make certain that his design will accommodate all its own waste sources satisfactorily.

9.3 Preliminary Site Investigations, Field Surveys and Instrumentation

Once the initial evaluation of site characteristics has been completed, the design objectives and constraints have been evaluated by the canal designer, and there is an agreement between the owner and the designer to proceed, the next step in the design process is a *preliminary site investigation*.

9.3.1 Objectives. If a proposed canal site has not been developed to any degree, and no canal channels have been constructed, the field measurement task is somewhat less complex than if a canal system is already in place and hydraulically connected to the receiving waterbody. In the latter case, in addition to the measurements required to establish the conditions existing at the site and in the receiving waterbody, it is necessary to evaluate the conditions in the existing canal system.

The investigator responsible for collecting information about the site, and the field data, should plan a preliminary site evaluation first, followed by one or more field surveys to obtain data for design. The objectives of the *preliminary site investigation* are:

- To determine the conditions which will limit field measurements at the site, such as overall depth of receiving waterbody, prevailing currents, proximity of dwellings, intensity of boat traffic, access to site from both land and water, and security at the site.

- To determine if any unusual conditions are present which will require additional measurements, special equipment, or special procedures.

- To determine the approximate range for the variables to be measured (which determines the type of instrumentation and support equipment that will be required).

- To locate elevation reference points for future surveying.

- To familiarize field crews with site logistics.

- To measure depths in canals for later determination of support equipment requirements.

The overall objective of the field surveys is to collect the data which will be required to

- Support the design analysis for the preparation of the trial canal system designs.
- Enable estimates to be made of calibrating coefficients for the numerical model.
- Obtain suitable data for model verification in the case of development of a numerical model.

The measurements needed to fulfill these objectives can be divided into long-term measurements and short-term measurements. The former require onsite installation of sensors and recorders at several locations. Structures placed in the water will require consideration of the range of water level over the period of recording, antifouling and anticorrosion precautions, and possible dynamic water loads during extreme weather events. All instruments will require periodic replenishment of the power source and recording media, and security.

Short-term surveys are those which involve intensive sampling from the shoreline and/or from a boat. It is necessary to plan at least two such surveys, to coincide with average climatic conditions in both the wet and the dry seasons, and preferably three or four surveys to obtain a more reliable sample.

9.3.2 The monitoring and sampling problem. The objective of taking measurements of any parameter which varies with location and time is to obtain, as completely as is practical, a series of three-dimensional pictures of the patterns of that parameter and other associated parameters over a period of time which includes at least the normal ranges of these parameters. For example, one would like to have a three-dimensional view of the water velocity, salinity, and water temperature, at intervals of one hour over a period of several weeks, together with a plot of wind speed and direction over the water for that period, and the dye concentration distribution resulting from a planned dye release. Aside from the obvious difficulty of interpreting such a large amount of data, it is not at all a practical objective because of the very large number of locations at which measurements would have to be taken, more or less simultaneously. Only a field survey with a complete set of automatic sampling instruments, either platform- or boat-mounted at each measuring point, would even approach such an ideal objective.

A field crew normally consists of one or two boats with a complete set of equipment and two or three persons per boat. If, for example, salinity, temperature, dye concentration and dissolved oxygen were to be sampled within a 2,000 ft canal, at 200 ft intervals along the canal, at three locations in each transect, and at the surface and at depth intervals of 1 ft to a total depth of 5 ft, there are 11 multiplied by 2 surface locations, by 6 depth locations, or 198 sampling points. Assume that the boat requires fifteen minutes at each location, including transit time to each new location, which implies that salinity, temperature, dye concentration and dissolved oxygen at each of six depths (twenty-four readings) can be made and recorded in fifteen minutes. It would still take eight and one-half hours (substantially more than one-half tidal cycle) to complete the measurements at all thirty-three locations. Such an approach certainly would not provide useful data, as there is too great a time span involved. Assuming, on the other hand, that two boats are available, that two variables are to be measured from each boat, and that only the eleven centerline stations are sampled, the total time required to sample the eleven stations would still be over an hour and a half.

It may be concluded that a field survey requires precise planning, reliable equipment, good training and considerable endurance if useful synoptic measurements are required over a several-day period.

When planning either a preliminary site investigation or a field survey, it is advisable to consider subcontracting field work to an organization which has had experience in coastal oceanographic and canal hydraulic measurements. The canal designer should satisfy himself that this organization has the proper instrumentation and equipment, and experienced field crews available, to be able to obtain and reduce the required data and to present that data in the desired form. If the owner and the canal designer decide that this is the most economical approach to obtaining the necessary information, the canal designer should prepare a detailed performance specification defining the variables to be measured, the locations of measurements, the frequency of samples, the total period of time required and the accuracy needed in the final results.

9.3.3 Measurement requirements for tidal canals. The information required for implementing a canal design cannot be obtained directly from discrete or continuous samples of the measurable variables. For example, the designer might wish to know about the water circulation patterns and dispersion characteristics in the receiving waterbody near the proposed entrance to the canal system. This information will have to be deduced from velocity profiles, salinity gradients, wind history, and dye concentrations, all limited by the amount and type of available instrumentation and the size of the field crew. Table 9.2 relates the desired information, information derived by engineering analysis and measured variables involved in canal design.

A calculation of the tidal prism is necessary for a given geometry and canal network layout because it enables some order-of-magnitude calculations to be made of mean velocities and tidal energy distribution. In order to calculate the tidal prism the geometry of the channels and the surface area of the system will have to be measured (if existing) or assumed. Also, the range of the tide at the entrance(s) will have to be known. In an existing system it is best to obtain an aerial photograph of the canal network for this purpose, at a scale of approximately 1:2,400 (1 in = 200 ft) or larger. In addition, it is necessary to survey the depths of channels and to take representative cross-sections for the tidal prism calculation.

Tidal elevation should be continuously recorded during the field survey. These data are needed for referencing channel depths, current velocity readings, and salinity, water temperature and water quality measurements taken either from a boat or from the shoreline. The tide recorder should be placed relatively near the measurement site.

Table 9.2 - Flow of Engineering Information From Measurements to Analysis.

Desired Information	Information Derived by Engineering Analysis	Measured Variables
Tidal Prism	Canal Geometry	Distances Depths Widths
	Tidal Elevation	Tidal Heights Tide Gauge Reference Level
Circulation Pattern	Water Surface Slope; Tidal Attenuation and Phase Shift	Tidal Heights at More Than One Location
Flow Q	Velocity Profiles Manual Integration	Water Velocity $u(x,y,z,t)$
Flow Q		Water Flow $Q(x,t)$
Circulation Pattern	Shear Velocity $u^*(x,t)$	Water Velocity $u(x,y,z,t)$
	Eddy Diffusivity Coefficients	
	Wind Plots	Wind Velocity (t)
	Salinity Profiles	Salinity (x,z,t)
	Temperature Profiles	Temperature (x,z,t)
Water Budget	Rainfall (area, time)	Rainfall
Runoff Hydrograph	Runoff (location, time)	Flow at Major Sources
	Infiltration Rate	Soil Samples
Pollution Source Location and Concentration	Pollutant Sources	
Water Quality Variations	Pollutant Distributions	BOD, pH, Coliforms, Turbidity, toxins
	D.O. Profiles	D.O. (x,z,t)
	Dispersion Coefficients; Dye Profiles	Dye Concentration $c(x,z,t)$
	Diffusion Coefficients	
Bed and Bank Stability	τ_o, τ_{bank}	Water Velocity
	Effective Grain Size	Grain Size
	$\tau_{critical}$	

The hydrodynamics or water circulation, both in the receiving waterbody and in the canal network, are reconstructed from measurements of current velocity, wind, temperature and salinity. Normally these data are taken at the centerline of a channel. Vertical velocity profiles are required for determination of the bottom shear stress, as well as for verifying the existence of layered flow and for deducing the relative influence of wind stress and salinity gradients on the overall circulation pattern. A detailed understanding of the circulation in a reach in which dye dispersion measurements are to be conducted is vital for obtaining useful dye concentration profiles. If it is found that the flow is not layered, and that conditions are well-mixed and velocity profiles are logarithmic from the surface to the bottom, simpler methods of velocity measurement (e.g., sampling the velocity at two or three depths) may be employed. It has been found, in Floridian canals, that such uniform conditions do not very often prevail.

If it is determined that wind velocity has a significant effect on the circulation, it is necessary, for verifying numerical model results, to have a continuous recording of the wind. The wind sensor must be placed at a height sufficiently above obstructions and sufficiently far from the water surface as to be relatively unaffected by local disturbances. The generally accepted height used by many investigators is 10 meters.

Temperature and salinity profiles over depth are required initially to ascertain whether the waters are well-mixed. These conditions should be checked during both ebb and flood tide, and at night as well as during the day. Analysis of these data, in conjunction with vertical current profiles and wind measurements, will indicate whether salinity and/or temperature gradients are affecting the flow.

The water budget at a given site, which is affected by rainfall, runoff, evapotranspiration and infiltration, is generally determined by analysis of existing data. However, there may be particular site characteristics which should be quantified for the canal design. The existence of salinity barriers and their elevation should be noted. Locations where natural swales or other possible local sources of freshwater could allow flow into the canal system should be evaluated to determine their possible effect on the structure of the density circulation.

Pollution sources near the site should be located and the constituents in these sources should be determined before a trial canal design is laid out. Information and data on conditions existing prior to alterations of the site will be required in a permit, and may indicate special precautions to be taken to prevent the introduction of existing pollutants into the system. Measurements of the pH, turbidity and the concentrations of suspected constituents such as fecal and total coliforms, biochemical oxygen demand (BOD), and toxins should be performed in a laboratory which uses standard methods and conforms to state regulations, since water quality can become a legal issue and is certainly a primary consideration in the permitting process.

Dissolved oxygen is one of the principal indicators of water quality. The measurement of DO profiles before any modifications to the site are made is necessary to establish baseline conditions. The effect of a particular canal design on DO is difficult to predict because of the variety of variables which can affect it. However, a canal system which has good fushing characteristics throughout, and which is designed to control pollutant inputs by adequate drainage and waste water handling, can be expected to be able to meet DO standards.

The flushing characteristics of an existing canal system may be investigated by means of dye studies. Properly conducted dye studies provide profiles of concentration over several tidal cycles, which may be used to calibrate the CANNET model for predictions of concentration distributions with time.

Bed and bank stability can be determined, for a given canal geometry, from soil samples. The samples should be large enough to supply material for both sieve analyses and hygrometer tests.

CHAPTER 10
INSTRUMENTATION AND DATA REDUCTION FOR CANAL MEASUREMENTS

The measurements needed for a canal hydrographic survey depend, to some extent, on the conditions and problems associated with a particular site. There are, however, certain common variables which will have to be quantified in everycase, and these will be described in this chapter.

Inherent in every measurement task are many considerations, details, and decisions that ultimately lead to a description of the physical, chemical, and ecological characteristics of the site, and of the canal system if one exists at that stage in the development. It is essential that the desired end results be carefully defined in as much detail as possible before any field work commences. The canal designer should be able to justify the need for each variable, and define the location, frequency, and duration of each set of required data. Furthermore, the required accuracy of each variable, and the form in which the results are to be analyzed and presented, should be clearly stated. Then, the considerations and decisions with regard to the instrumentation and data reduction procedures can be made with a reasonable opportunity for a successful field survey program.

10.1 Desirable Specifications for Canal Instrumentation

A variety of instruments and other equipment is available for measuring the variables required for the engineering analysis that leads to a trial canal design. In selecting an instrument for a particular measurement, however, there are a number of important considerations which determine its suitability for the field conditions inherent in coastal survey work. Desirable specifications for an instrument or piece of equipment for this application include:

- *Accuracy*: 1 to 5 percent full-scale is adequate for most measurements. Usually instruments with analog meters for readouts will have no better than 2 percent accuracy. Digital readouts are preferable provided they can be easily read in bright sunlight.

- *Reliability*: Best available is required. The high cost of a field survey requires that all equipment be operational for the entire period of the survey.

- *Power source*: 12 volts DC, with sufficient regulation and protection for operation from lead-acid batteries, or as a second alternative, operation from built-inrechargeable cells with low power consumption, is required. Operation on 12 VDC is preferable if the equipment is to be used in a boat which has an alternator on the engine, since then recharging is relatively convenient during and at the end of the work day. If only 110 VAC operation is possible, an inverter is preferable over a generator because the noise and exhaust from the latter are difficult to work with and may adversely affect the crew's performance.

- *Environmental conditions*: Salt moisture, direct sunlight, rain squalls and relative humidity approaching 100 percent at night are some of the conditions that must be planned for. Equipment with mechanical parts require special precautions to protect against sand and corrosive moisture. Equipment should be relatively light and portable since often it must be carried to the site by hand.

- *Simplicity of operation*: For many reasons, simplicity of operation is an important attribute of field equipment. New field personnel without thorough training will inevitably have to use the equipment, and even experienced field personnel at times find themselves busy and concerned about many minor difficulties. A certain level of complexity is inherent in any piece of equipment, but special attention to the arrangement and operation of support equipment can greatly simplify procedures.

- *Calibration*: It is desirable that simple, direct means of calibration either be built into each piece of equipment, or made available in the support equipment. This assures more reliable operation, and usually serves as a check on whether the equipment is functioning properly or not.

- *Recording*: Where possible, arrangements should be made for recording data against a reliable time base, with some means available for recording notes by the operator at the time measurements are being made. This is particularly true when measurements are taken from a moving boat. Automatic locating and recording systems for boat position are not usually available for survey boats, so manual recording of navigational position on a strip chart is often the only practical solution.

10.2 Instrumentation and Support Equipment Used by the Hydraulic Laboratory

In this section the major instrumentation and support equipment used by the Hydraulic Laboratory for canal survey work will be described. Manufacturer's names and equipment model numbers are included to serve as guidelines for those planning to do similar work. Table 10.1 summarizes the types of equipment and instrumentation which can be used for each of the measured variables listed in Table 9.2.

10.2.1 Tide recording. A reliable tide recorder is essential for canal survey work. A number of different water level recorders are available and suitable for this application. Recording is available in a variety of formats, either continuous or discrete in time. A recorder with a spring-driven clock and fully-mechanical mechanism has the advantage of not requiring

Table 10.1 - List of Field Equipment and Instrumentation That Can be Used for Each of the Measured Variables in Table 9.2.

Measured Variables	Methods of Measurement	Field Equipment or Instrumentation
Distance	Range	Optical Range Finder
	Direct	Surveying Tape
Depth	Direct	Graduated Pole
	Direct	Graduated, Weighted Line
	Acoustic	Depth Recorder
Canal Width	Range	Optical Range Finder
	Direct	Graduated Line, 100 ft
Tidal Height	Tide Staff	Graduated Pole, Stilling Well
	Float	Tide Recorder, Stilling Well
Water Velocity	Propeller	Various
	Savonius Rotor	Various
	Electromagnetic	Two-axis Electromagnetic
Water Flow Rate	Integrated Velocity	Velocity Meter
	Direct	Transport Integral Device
Wind Velocity	Anemometry	Anemometer With Recorder
		Hand Held Anemometer
Salinity	Titration	Titrating Kit
	Conductivity	Electronic Meter
Temperature	Direct	Thermometer
	Direct	Thermistor and Electronics
Rainfall	Direct	Recording Rain Gauge
Runoff	Flow	Flowmeter
Infiltration	Soil Sample	Sampler
Pollutants: BOD	Laboratory	Sample Bottles
pH	Laboratory	Sample Bottles
Coliforms	Laboratory	Sample Bottles
Turbidity	Laboratory	Sample Bottles
Toxins	Laboratory	Sample Bottles
Dissolved Oxygen	Titration	Titration
	Diffusion	Electronic Meter
Dye Concentration	Direct	Fluorometer and Pump
Grain Size	Soil Sample	Sample Bottles

electrical power, but it does require replenishment of the recording media at regular intervals. The Hydraulic Laboratory has found the Leupold Stevens Type F, Model 68, to be extremely reliable over long periods of time. This instrument features a choice of elevation and time ranges, by means of replaceable gears, and an accurate eight-day spring-driven clock. Thus, no external power is required.

A recording tide gauge will require a stilling-well for dampening wave action, installing the recorder, and providing security for the equipment. Actual techniques for installing the equipment are dependent on the physical arrangement of the stilling-well and depend to a great extent on the length of time the gauge will be installed. The structural requirements should be determined for the length of time the gauge will be installed, to withstand expected climatic conditions and natural and man-caused interference, and to provide security for the equipment. The stilling well design used by the Hydraulic Laboratory is described in Section 12.4.5 in Morris, Walton & Christensen [1978].

The tide record should be referenced to mean low water (MLW), particularly if two or more gauges have to be referenced to a common datum. MLW also serves as a common reference point for repeated or supplementary surveys taken at different times at the same site. Tidal data used in preparation of a hydrographic survey which supports a permit application must be referenced to MLW, and such a reference would be required if the data were ever contested in legal proceedings. The only time the tide record need not be referenced is for purposes of a preliminary site evaluation, in which the approximate tidal range at the site is being confirmed. It is not possible to determine MLW from an unreferenced record to any acceptable degree of accuracy because the tidal elevations are subject to a variety of offsetting factors, including bottom topography and local storms.

A tide gauge may be leveled in to the geodetic net, which has been extended to most coastal areas of Florida, by extending a survey to the nearest benchmark in accordance with procedures described by Piccolo [1976].

10.2.2 Distance measurements. Distances on the ground may be obtained by means of an optical range finder, although direct measurement is much more accurate. Discrepancies in the optical results of as much as 20 percent have been experienced by the Hydraulic Laboratory. Direct measurements by means of a 100 ft fiberglass surveyor's tape have been found to be relatively easy and well within accuracy requirements. The fiberglass open reel tape is preferable over steel tape because it is little affected by saltwater and sand.

10.2.3 Depth recording. A Benmar echo sounding recorder, Model DR-68, is mounted in the Hydraulic Laboratory's survey boat for taking depth profiles. The instrument is powered by 110 VAC, so an inverter has been provided to convert the 12 VDC boat supply for this purpose. The recorder's electronic components are sealed, and the instrument is designed for work in a salt moisture environment. Adjustments enable good recordings of the bottom to be obtained in depths of about 3 ft or greater on a scale of 50 ft. The chart is calibrated before each run by means of a 10 ft pole graduated in tenths of feet. The time and location of the boat should be noted on the depth recorder chart at regular intervals.

The manufacturer provides no quantitative statement as to the accuracy of the instrument. The accuracy is actually limited by the data analyst's skill in distinguishing the hard bottom from the silt layer, and the first reflection from other reflections. The instrument has an absolute accuracy within ± 0.1 ft in the first 10 ft of depth, as verified in the field.

10.2.4 Current measurements. An Ott laboratory propeller meter, Model C.1, can be used to measure tidal current speed. However, this instrument has serious limitations in the field. First, it has a threshold of about 0.16 fps and is, therefore, completely unresponsive to typical canal currents, which are normally less than 0.1 fps. The instrument also must be hand-held and pointed in the desired direction, and since canal waters are not clear beyond several inches, it is difficult to ensure that the propeller is directed exactly along the centerline of the canal, even when suspended by a long metal wading rod. In addition, it is a pulse counting system which requires that the reading by integrated over at least a minute, and often several minutes if signficant differences in successive pulse counts are obtained. A Savonius-rotor current meter may be available with a lower threshold, perhaps down to 0.1 fps, but it too has no directional capability. A Cushing dual-axis electromagnetic current meter, Model 82-CP velmeter sensor with Model 632-P portable converter will provide the necessary low range and directional capability. This meter consists of a probe, 3/4 inch in diameter and 11 inches overall, connected by cable to a signal converter mounted in a hermetically sealed, internal-battery-powered, meter box. The probe measures the instantaneous longitudinal and lateral velocity components at a point 3 inches from its tip by detecting the amount of distortion that is caused by the flow of water in a uniform magnetic field generated at this section on the probe. Each of the two velocity components is displayed on one of two deflection meters, on a scale calibrated in fps. The full scale can be changed to as low as 0.3 fps, with graduations at 0.01 fps. The output time constant (integrating time) associated with a reading can be changed over a range of 0.1 to 10 seconds. The manufacturer specifies the linearity of the system to be ± 1 percent of full scale, the zero offset to be ± 0.01 fps or better, and random noise (rms) to be $0.002/\sqrt{T}$ fps, where T is the output time constant in seconds. For operation with a time constant of one second, this is an overall accuracy of ± 0.012 fps or better. The threshold of this meter is about 0.01 fps and the overall accuracy, using the panel meters, is about ± 2 percent.

Since the probe is extremely sensitive to any movement, a portable tower can be built, as shown in Figure 10.1, for use in canals. This tower, which can be adjusted in length from 11 to 15 feet, is placed in the canal at a predetermined location with the probe holder pointing toward the shore, and guyed securely by a three-point arrangement to the shore. A carriage holding the velocity probe is free to slide up and down the tower, enabling the probe to be set any desired depth. With the probe oriented vertically both longitudinal and transverse current components can be continually monitored. The probe must also be oreinted around its axis so that the directions of the velocity components are known. In order to avoid the possibility of not recording the direction the probe is pointed, a standard procedure has been adopted in which the "north" pole of the probe is always pointed toward the dead-end corresponding to positive flow in a direction out of the channel. The monitoring is performed on the bank of the canal or on the work boat. A ± 5 VDC signal is provided for connection to a strip chart recorder or a data acquisition system.

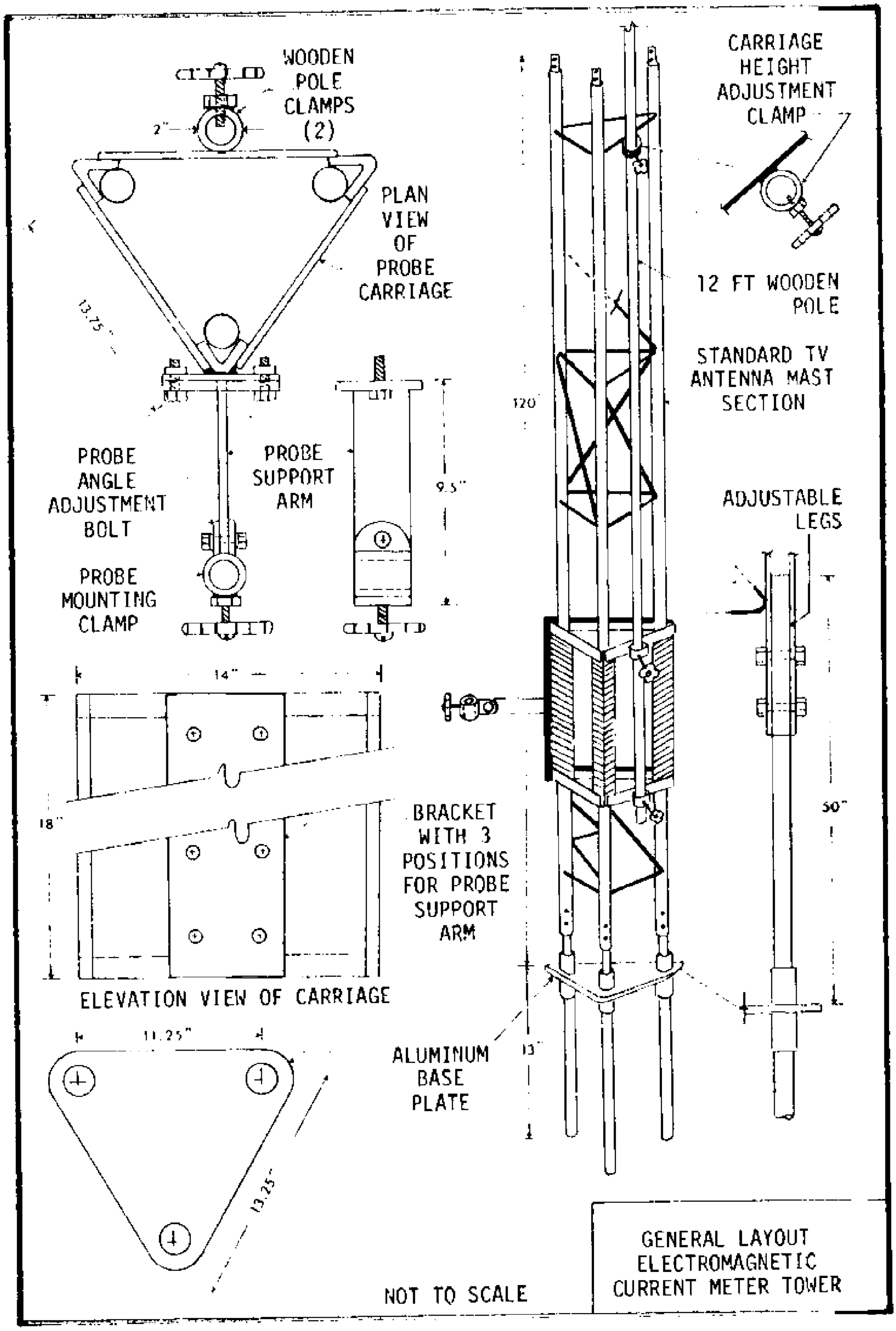


Figure 10.1 - Dimensions of Velocity Meter Tower With Adjustable Carriage, Designed and Built by Snyder Oceanography Services.

10.2.5 Wind recording. A Davis hand-held low speed anemometer, Model A/2-4", can be used to obtain samples of wind speed. This is a useful meter, but it has the disadvantage that it can only be held about 10 feet off the surface of the water. A measurement taken this close to the water is a measurement of a local wind component which has been affected by the presence of trees and structures in the vicinity of the site, and is therefore not representative of the wind system prevailing over the whole canal. In addition, the hand-held meter is not directional, and it cannot provide the long-period recordings (four to five days continuous) needed to evaluate the effect of winds on the circulation.

An R. M. Young windvane and three-cup anemometer, Model 6001, with a Model 6420J recorder/translator, is used by the Hydraulic Laboratory for long-term wind recordings. Mounted on a ten-meter-high telescoping antenna mast, the sensor is well above the tops of most trees that grow along canal banks. The mast can be set up by two persons in about forty-five minutes, and a hand-held compass used to adjust the readout for the direction of the windvane after the mast has been erected and guyed. Wind speed and direction are sampled every two seconds and recorded on a Rustrak recorder, part of the recorder/translator unit which has been mounted in a waterproof plexiglas case which in turn is attached to the mast after it has been erected.

The windvane threshold is below 1.6 mph and is unfiltered. The cup anemometer threshold is also below 1.6 mph, but this signal is filtered with a fifteen-second time constant to reduce scatter during gusty winds. The wind speed (low) scale is 0 to 50 mph, and the resolution is stated by R. M. Young to be better than 5 percent (2.5 mph). The meter will be calibrated in the factory before delivery, but should be recalibrated before each use.

10.2.6 Salinity measurements. Salinity measurements have been obtained both by titrating water samples with a LaMotte test kit Model POL-H code 7459, using the Harvey adaptation of the potassium chromate-silver nitrate reaction, and by means of a LaMotte conductivity meter, model DA. Considerable variation has been found in the results indicated by the two methods, and it has been concluded that this conductivity meter is not reliable enough for field work. Measurements by titration are accurate to about $\pm$ 1 ppt.

Salinity is not measured directly by conductivity measurements. The salinity of seawater is approximately the mass ratio of total dissolved solids to the total sample of water. Because the composition of pure seawater is almost completely uniform throughout the oceans, the salinity of pure seawater can be expressed in terms of the conductivity of the ions in solution and water temperature. However, if the seawater or canal water contains other ions in solution, such as sulfides, the conductivity will be affected and the reading will not be convertible to a true salinity measurement.

In general, the conductivity method is faster and more convenient for salinity measurement than the titration method, even though a simultaneous temperature measurement and a table are required for conversion to salinity from conductivity. The principal advantage is that it is far easier to lower the conductivity and temperature probes to the desired depth than it is to bring up a water sample, pour it off into a sample bottle and then perform the various steps of the titration. Reliable conductivity instruments are readily available, some with automatic temperature correction, but these should always be supplemented by as many titrations as possible to insure backup readings in case the conductivity meter does not operate properly or in case the water is not pure seawater, and for calibration purposes.

10.2.7 Water temperature measurements. Water temperature measurements have been obtained both from a mercury thermometer and from a LaMotte temperature meter Model KA. The readings obtained by these two methods compare within two degrees in the field. Since temperature variations with depth have never been found to be greater than a few degrees, and each measurement can be completed quite quickly, both are considered suitable for measurements of temperature gradients in canals.

10.2.8 Dissolved oxygen measurements. Dissolved oxygen can be measured either by titration or by means of a polarographic/membrane probe and associated electronics. The titration method on the whole is simpler, easier to learn to use, and more reliable. It, therefore, is generally preferred by field crews. The electronic instrument has certain advantages over titration and, once mastered, provides more measurements per unit time.

For the titration method, water samples are first collected from the desired depth with a Van Dorn bottle. Then the sample is drawn off through the valve in the bottle into a small (75 ml) sample bottle, which is filled to overflowing and capped. The operator should be careful not to entrain oxygen in the sample as it is poured into the empty sample bottle.

A Yellow Springs Instrument Co. Model 57 dissolved oxygen meter can be used to obtain vertical dissolved oxygen profiles. Calibration of the meter and the probe are straightforward, but depend upon instrument position, atmospheric pressue and proper warm-up (about fifteen minutes) of the system. A correct measurement cannot be made until the salinity adjustment is set. The reading is automatically compensated for *in situ* temperature, which can also be read on the front-panel meter. The probe must be agitated while the measurement is being made to insure that bubbles do not lodge on the membrane and that the sample is flushed continuously across the sensor.

10.2.9 Dye concentration. The Hydraulic Laboratory uses a Turner Designs Model 10-005 field fluorometer for measurement of the concentration of the fluorescent water-tracing dye Rhodamine WT. The operation of a fluorometer is based on the principle that a fluorescent material has the ability to absorb light at one wavelength and respond almost instantly with the emission of light at a new and slightly longer wavelength. The intensity of the emitted light is proportional to the concentration of the fluorescent material present in the sample. The fluorometer, making use of this property, irradiates the sample through a filter designed for the particular type of dye being used, and measures the intensity of the emitted light through another filter, adjusting the measured value in accordance with the range, sensitivity and blank settings on the instrument. The reading on the meter, in the range 0 - 100, when divided by the range switch position, the setting of the range multiplier, and a calibration constant gives the numerical value of the concentration.

The highest concentration which the instrument can measure without a calibration curve is about 10^{-7} (0.1 ppm), and its sensitivity is limited to concentrations of 10^{-11} (ten parts per trillion) of Rhodamine WT, due to electrical noise. The manufacturer's specifications state that its linearity is within $\pm$ 1 percent and its resolution is to $\pm$ 0.5 percent of full scale. The instrument provides analog outputs proportional to the reading of concentration and the range setting, and an output representing the setting of the range multiplier, suitable either for recording on a strip chart recorder or in a data acquisition system. One particularly useful feature is the automatic range change over two ranges, 10^{-7} to 10^{-9} and 10^{-9} to 10^{-11}. The change of range from one of these two automatic ranges to the other is accomplished by means of a manual switch. The fluorometer can be arranged either to measure the concentration of a sample in a cuvette, or of a sample pumped continuously through the machine. The fluorometer has been used successfully in both modes in the field.

When used at a station to measure the concentration of dye at various depths, it is sometimes convenient to set the instrument up for individual samples which have been taken in water sampling bottles. This is particularly true when other measurements are to be obtained from the water samples at the same time, such as salinity, temperature, or dissolved oxygen. More often, however, a continuous sample is desired at a specific location. Such a sample can be pumped up a hose and through the fluorometer with the continuous sampling cuvette installed.

A sampling system in its simplest form consists of a set of hoses and a pump, as shown in Figure 10.2. It is recommended by the manufacturer that the intake hose be connected directly to the flurometer, and that the fluorometer be connected directly to the suction side of the pump to avoid the introduction of air bubbles into the sampling chamber of the fluorometer.

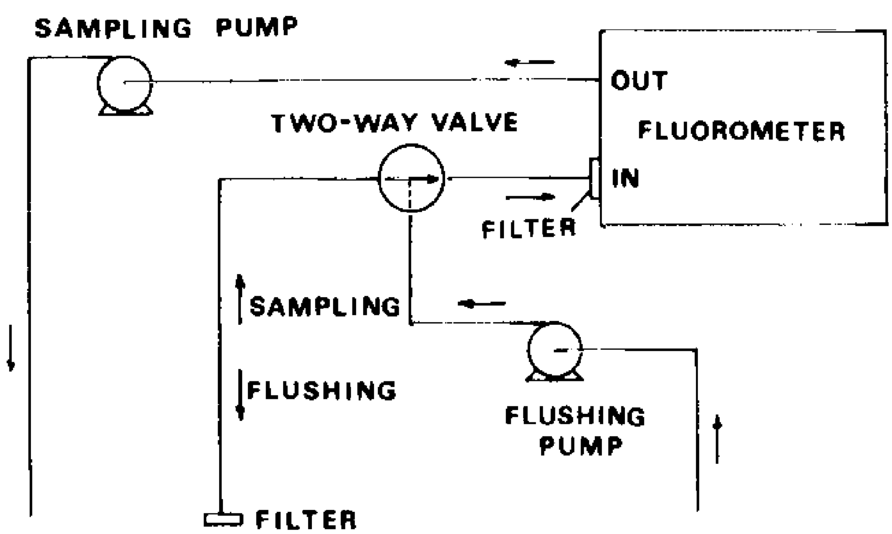


Figure 10.2 - Diagram of Continuous Flow Dye Sampling System.

The manufacturer additionally cautions against the use of vinyl tubing, which tends to absorb minute quantities of dye, and stresses that the hose or tubing must be completely opaque for a distance of at least 2 ft on both sides of the fluorometer. It has been found that rubber garden hose is adequate for this application, although there can be some difficulty in maintaining air-tight connections. Higher quality commercial hose and quick-connect couplings have been found to provide the most convenient setup, and most reliable operation.

The sampling pump should be limited to less than 5 gpm, according to manufacturer's specifications, and must operate from power supplies available in the field. Two models of Jabsco pumps have been found to be adequate and reliable: the Model 6360-0001 self-priming 12 VDC "bronze water puppy", and the Model 8860-0001 self-priming 12 VDC "bronze mini puppy". These provide a flow of 4.6 gpm at 10 ft head and 1.4 gpm at 10 ft head, respectively.

The details of some boat-mounted dye sampling systems are described in Morris, Walton & Christensen [1978, pp. 525-529].

Standardization of the fluorometer is a necessary task in the field. In standardization, the span control is set at a reading of ten on the meter for a standard concentration of 10^{-9} (1 ppb) on the mid range settings (X100, X1) and the blank, which subtracts any background fluorescence in the water, is set for a zero reading. Additional information on standardizing the fluorometer, and interpreting its readings, will be found in Morris, Walton & Christensen [1978, p. 529 and pp. 534-539].

CHAPTER 11
CANAL DESIGN ELEMENTS

Canal design is an iterative process that begins with the layout of a trial canal network. A trial network design is a canal system layout that both conforms to the site, and meets the identified design objectives, guidelines, and constraints. The trial canal design involves drawing the outline of the canal system on a topographic map of the site, and calculating the specific values of the independent variables for setting up the numerical model to be tested.

In order to synthesize a trial canal design, the canal designer needs to have an understanding of the operating characteristics of the various design alternatives available. One way to obtain this understanding is to have previously studied and tested a variety of complete canal networks and to use this experience as a guide. A second, and more practical approach is to visualize a canal network as a system assembled from a variety of simpler parts, which may be called *elements*. For example, an element could be a straight, dead-end finger canal, or a finger canal terminating at a boat basin. A more complex element could be a straight, dead-end canal with several parallel finger canals oriented at ninety degrees to it. If the characteristics of these kinds of elements can be described quantitatively, it is reasonable to expect that these characteristics, by the principal of linear superposition, will also apply in a network made up of such elements.

Some of the characteristics of canal design elements may be described in terms of their *sensitivity*. This is the relative change in the magnitude of a particular variable, such as the concentration in a canal, that results from a specific change in the magnitude of a certain design variable. For example, the change in the concentration distribution for a given change in the width of the canal, measured after 50 tidal cycles, would provide the canal designer with some quantitative guidance when he begins to synthesize a trial canal design. The applicability of the results of sensitivity analyses to canal network design is, however, definitely limited. The physical phenomena associated with mass transport in a tidal canal network, the tides, wind, secondary currents, and salinity gradients, interact to different degrees at different locations in the network and at different times. Therefore, the only reliable prediction that can be obtained is a simulation of all the defined variations in each variable over the entire time span of interest. The concept of design elements, therefore, can provide only guidelines to the initial or trial design.

11.1 One-Dimensional Sensitivity

The sensitivity of a one-dimensional, prismatic, trapezoidal, dead-end canal may be tested by establishing a set of design values and varying each of these, one at a time, to find the resulting equilibrium concentration profile. To do this, a one-dimensional computer model has been used with a sinusoidal tide at the entrance. Since the resulting concentration profiles at each computational node in the canal also vary sinusoidally with the tide, the envelope of the concentration values is observed until the concentration at any node does not change by more than some specified percentage from one tidal peak to the next. The model, during this time interval, is reaching a numerical equilibrium as it iterates in time through successive solutions.

The computational boundary conditions at the dead-end and at the tidal entrance have been established as a result of a joint study conducted by the Hydraulic Laboratory and the Martin-Marietta Corporation, using a large hybrid computer [Morris, Walton & Christensen, 1977]. It was found that, at the dead-end of the canal, the concentration was best described by a linear extrapolation of the concentration at the two adjacent nodes:

$$c_0 = 2c_1 - c_2 \tag{11.1}$$

where the subscript refers to the number of the computational node, zero at the dead-end.

At the tidal entrance a dual-boundary condition was established. During the ebb tide the concentration at the entrance was calculated from the concentration at the adjacent node inside the entrance, by using a backward difference,

$$\frac{c_{TE}^{n+1} - c_{TE}^{n}}{\Delta t} = -\frac{u_{TE}^{n}\, c_{TE}^{n} - u_{TE-1}^{n}\, c_{TE-1}^{n}}{\Delta x} \tag{11.2}$$

where Δt = computational time increment, [T]; Δx = computational space increment, [L]; n = current time level, [T]; TE = tidal entrance node number. Assuming that the concentration at the tidal entrance reaches a value of c_{LT} at low tide, a first-order decay was used to describe the time history of the concentration at the tidal entrance during the flood tide:

$$c_{TE} = c_{RW} + (c_{LT} - c_{RW}) \exp(-3t'/\tau) \tag{11.3}$$

where c_{RW} = concentration in the receiving waterbody, (dimensionless); t' = elapsed time since low tide, [T]; τ = time decay coefficient, [T]. This form of the boundary condition produces a concentration at the tidal entrance within two percent of the background concentration, c_{RW}, after τ units of time following low tide.

The design values used for the sensitivity tests in the prismatic, trapezoidal, dead-end canal are summarized in Table 11.1. The results of sensitivity tests on eight canal variables are shown in Figures 11.1a through 11.1f. Two of these parameters, the tidal amplitude a, and the entrance decay coefficient τ, which would be established by measurements at a particular site, are considered to be "non-controllable". The other six, canal length L, bottom width b, inverse side slope s, mean tidal depth d_o, equivalent sand roughness k, and dimensionless longitudinal dispersion coefficient K, are considered to be "controllable" design variables. The results of these sensitivity tests are summarized in Table 11.2.

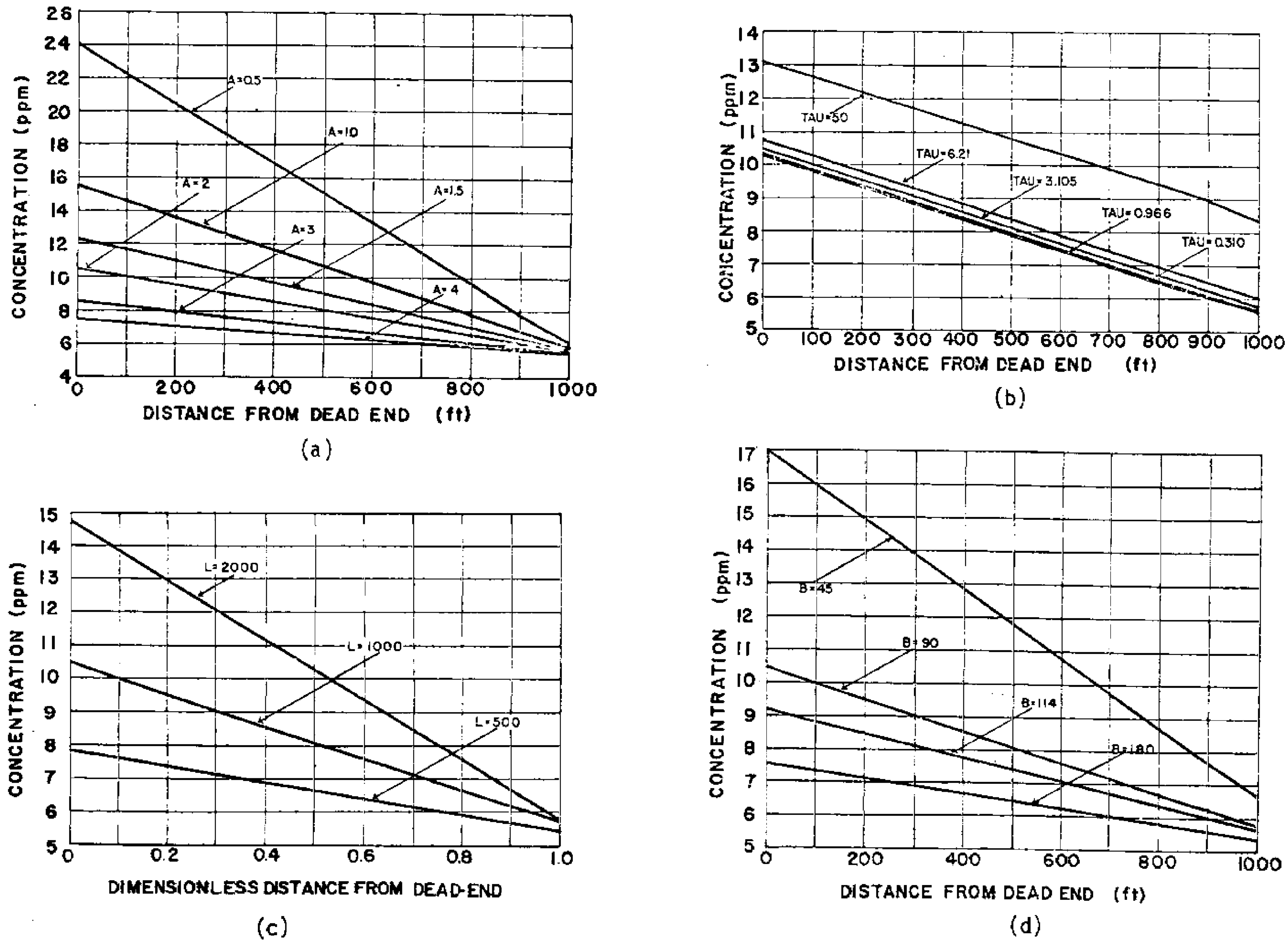


Figure 11.1 - Sensitivity of Dead-End Equilibrium Concentration for (a) Tidal Amplitude, A; (b) Entrance Decay Coefficient, τ; (c) Canal Length, L; and (d) Bottom Width, B. A, L and B are in ft. TAU is in hrs.

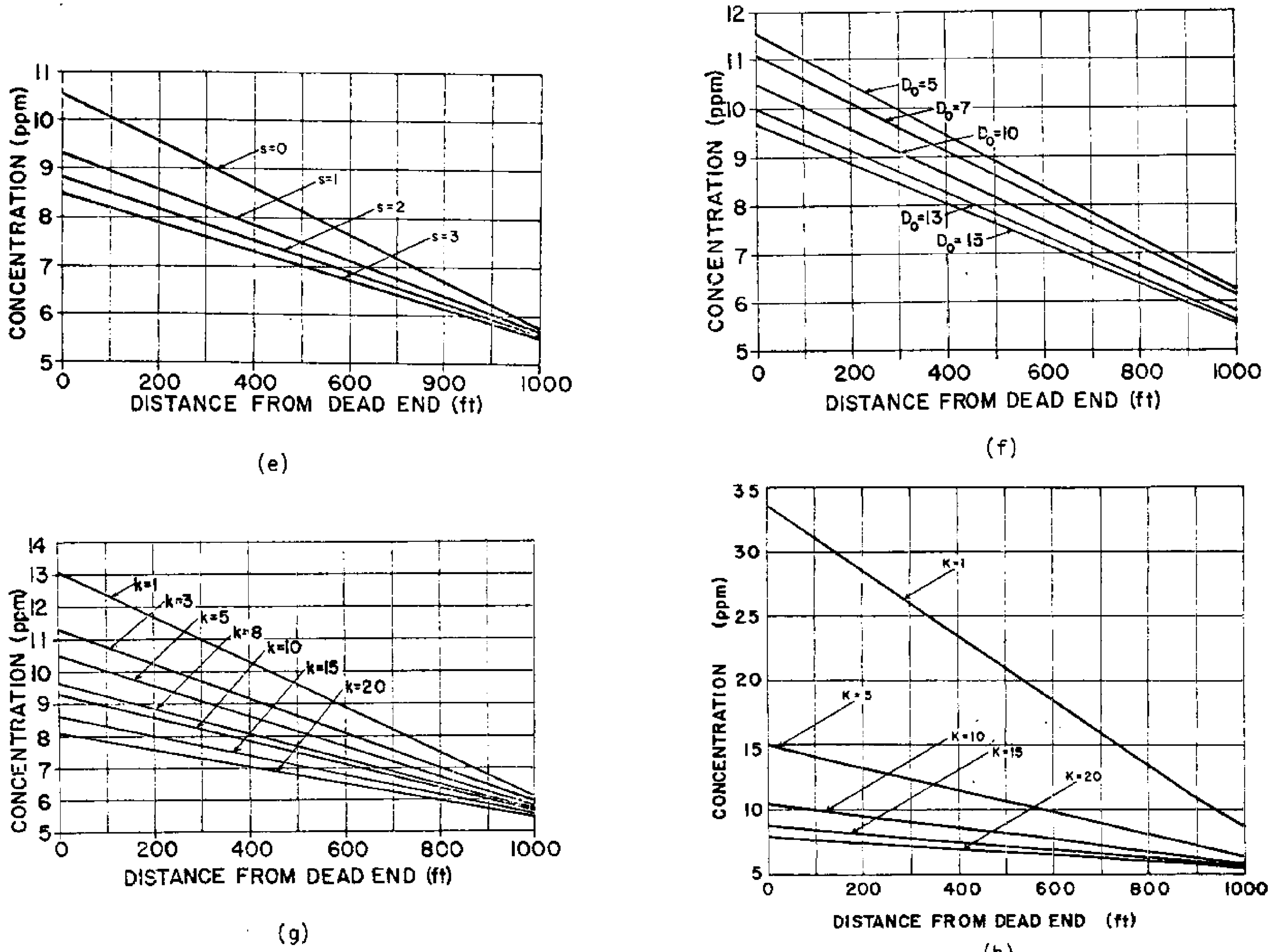


Figure 11.1 (cont'd) - Sensitivity of Dead-End Equilibrium Concentration for (e) Inverse Side Slope, s; Mean Tidal Depth, D_o; (g) Equivalent Sand Roughness, k; (h) Longitudinal Dispersion Coefficient, K. D_o and k are given in ft.

Table 11.1 - Design Values Used for One-Dimensional Canal Sensitivity Analyses.

Canal Design Variable	Value
Tidal amplitude, a	2 ft
Entrance decay coefficient, τ	6.21 hr
Canal length, L	1000 ft
Canal bottom width, b	90 ft
Inverse side slope, s	0 ft/ft
Mean tidal depth, d_o	10 ft
Equivalent sand roughness, k	5 ft
Longitudinal dispersion coefficient, K	10 --
Tidal period, T	12.42 hr
Lateral inflow, q_I	0.04 ft^3/hr/ft
Concentration of lateral inflow, c_I	100 ppm
Background concentration, c_{RW}	5 ppm

Table 11.2 - Relationship and Sensitivity of Dead-End Equilibrium Concentration to Changes in Canal Design Variables in One Example of a Prismatic, Dead-End Canal.

Canal Design Variable	Relationship	Sensitivity
Tidal range, a	decay	high
Tidal entrance decay coefficient, τ	positive linear	low (for $\tau \leq 6.21$ hr)
Length, L	positive linear	high
Bottom width, b	growth	high
Inverse side slope, s	decay	low
Mean tidal depth, d_o	negative linear	low
Equivalent sand roughness, k	decay	moderate
Longitudinal dispersion coefficient, K	decay	high

The "relationships" in the second column of Table 11.2 are designated "linear" if the change in concentration with changes in the parameter are within a $\pm$ 2 percent band on either side of a straight line. A linear relationship increases the concentration with an increase in the value of the variable. "Negative" refers to cases where the concentration decreases with an increase in the variable. A "decay" relationship is one in which the negative slope of the curve giving the concentration as function of the considered parameter is approaching zero the parameter increases. The "growth" relationship is opposite to a decay relationship. The notations under the heading "sensitivity" are qualitative judgements based on the results in Figures 11.1a through f.

Two of these sensitivities require additional comment. The relationship for the entrance decay coefficient, τ, shows that this variable has only a minor effect on the equilibrium concentration in the canal, provided its true value is a half-tidal cycle or less. This implies that in many simulations the value of this decay coefficient will have little effect on concentration predictions.

The sensitivity of the equilibrium concentration to the value of mean tidal depth would appear, once and for all, to solve the question about the proper depth for a canal. Unfortunately, this result is somewhat misleading. For instance, the value of the dimensionless dispersion coefficient K is a function of the shape and form of the channel and, therefore, the lateral velocity gradient, which in general will decrease with depth, thus decreasing dispersion. Also, too great a depth can exceed the depth of penetration of light, which is required for bottom vegetation. Therefore, because the one-dimensional model omits other phenomena that are significant in the dispersion and mixing process, an evaluation of the proper depth for a canal network should be based on results obtained from a multi-dimensional model, such as CANNET2D or CANNET3D.

11.2 Two-Dimensional Sensitivity

Having, from Section 11.1, some preliminary indications of the sensitivity of flushing in a prismatic, dead-end canal, it is appropriate to consider elemental canal systems. A simulation of the concentration changes in a canal system or network requires the solution of the mass transport equation, which can be accomplished with the CANNET canal network model. Since this model incorporates the effects of variable winds, variable freshwater or pollutant inflows, bends in the channels (3D version only), movement of a saline wedge, and a boat basin located at the end of a finger canal, it will be possible to consider the sensitivity of concentration to some of these variables as well, with this model.

11.2.1 Sensitivity of prismatic canal. To study the effects of changes in wind speed and mean tidal depth on a trapezoidal, prismatic channel, a series of tests were conducted on a 2000 ft long canal, using the two-dimensional canal network model CANNET2D. The wind was directed into the canal, and its speed varied between zero and five mph for different runs. The uniform mean tidal depth of the canal was adjusted between four and twelve feet for most of the wind speeds. At the beginning of each simulation the canal was uniformly loaded with the same mass of pollutant, which resulted in a different initial concentration for each different value of mean tidal depth. The simulation was then run with a constant wind and no pollutant inflow for 50 tidal cycles, and finally the mean value of concentration in all of the computational cells in the model was determined as an indication of the remaining mass of pollutant in the system. Figure 11.2 shows the mean value of concentration after 50 tidal cycles, c_{50}, divided by the initial concentration, c_i, which is a measure of the flushing effectiveness of the canal. A low number for the ratio c_{50}/c_i indicates a high flushing capability.

It can be seen that, for zero wind, the best depth for flushing is the *smallest*, but that as the wind is increased to three mph, the best depth for flushing is the *greatest* depth. Since the wind is held constant in speed and direction over the full 50 tidal cycles these solutions are not realistic, but they do illustrate some basic principles which may be described on a qualitative basis.

11.2.2 Some canal system design elements. Having to this point discussed only the sensitivity of individual dead-end canals, it is appropriate to investigate some of the features of systems of canals. To be precise in the nomenclature, it will be useful to define a *system design element* as a simple system of canals that can be used as a basic building block in a larger system, or network, and a canal *network* as an interconnected set of design elements specific to a particular site. Thus, design elements will be simple configurations amenable to generalizations about sensitivity and flushing, whereas networks can take many different forms and will be too variable to generalize.

Three different canal system design elements were devised to test the sensitivity of flushing for changes in wind, the addition of a basin, and the addition of a second tidal entrance. These configurations are shown in Figure 11.3. The dimensions of the finger canals, and their spacing, were obtained by laying out 100 by 200 foot lots in a typical residential finger canal arrangement (Figure 11.4), and specifying trapezoidal canals with the dimensions given in Table 11.3. The other parameters used in the numerical model for these simulations are also included in Table 11.3.

For these tests the wind, held constant throughout each simulation, was directed along the centerline of the canal either toward the dead-end (a west wind) or toward the entrance (an east wind). The uniform background or receiving water concentration, c_{RW}, throughout the canal was initially set either at zero or at one ppm, and the lateral inflow was set at a constant value of 0.04 ft^3/ft-hr and 100 ppm concentration into the surface layer. The values of the dispersion coefficients and the vertical momentum transfer coefficient were set at the values found by calibrating the model with South Florida data.

With a steady inflow of pollutant mass, it is anticipated that the concentration in the system will increase rapidly at the beginning of the simulation, and assymptotically approach an equilibrium condition. It is of interest, then, to compare the concentration differences after a fixed number of tidal cycles to determine the relative sensitivity of the rate of flushing to wind and to increases in the tidal prism. The two-dimensional version of the model, CANNET2D, was used with three layers for all of the tests described below.

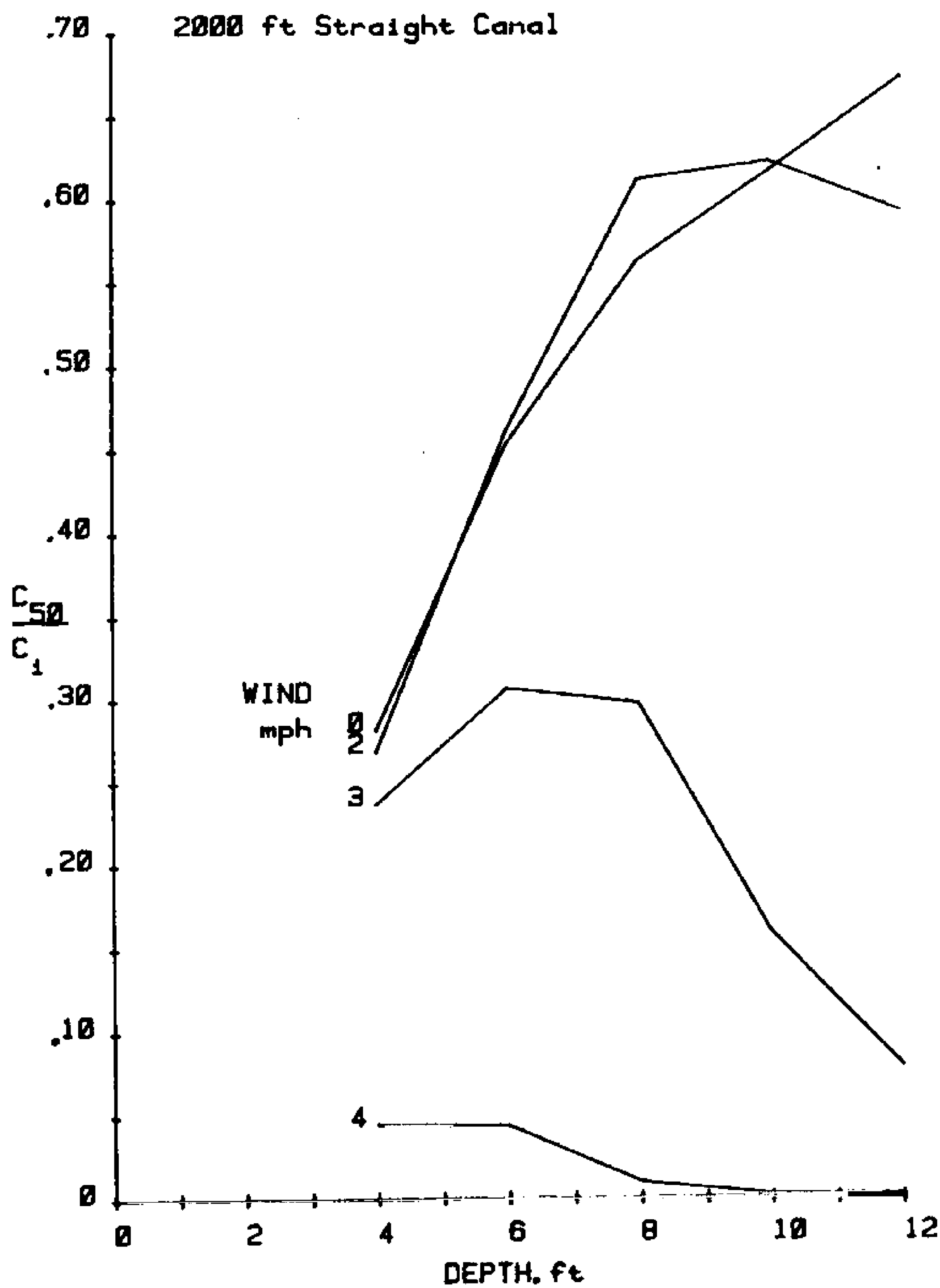


Figure 11.2 - Sensitivity of a Prismatic Canal to Changes in Wind Speed.

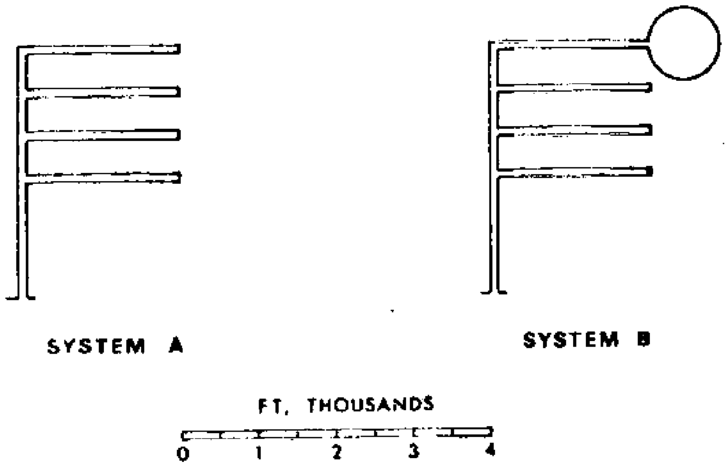


Figure 11.3 - Canal System Design Elements A and B.

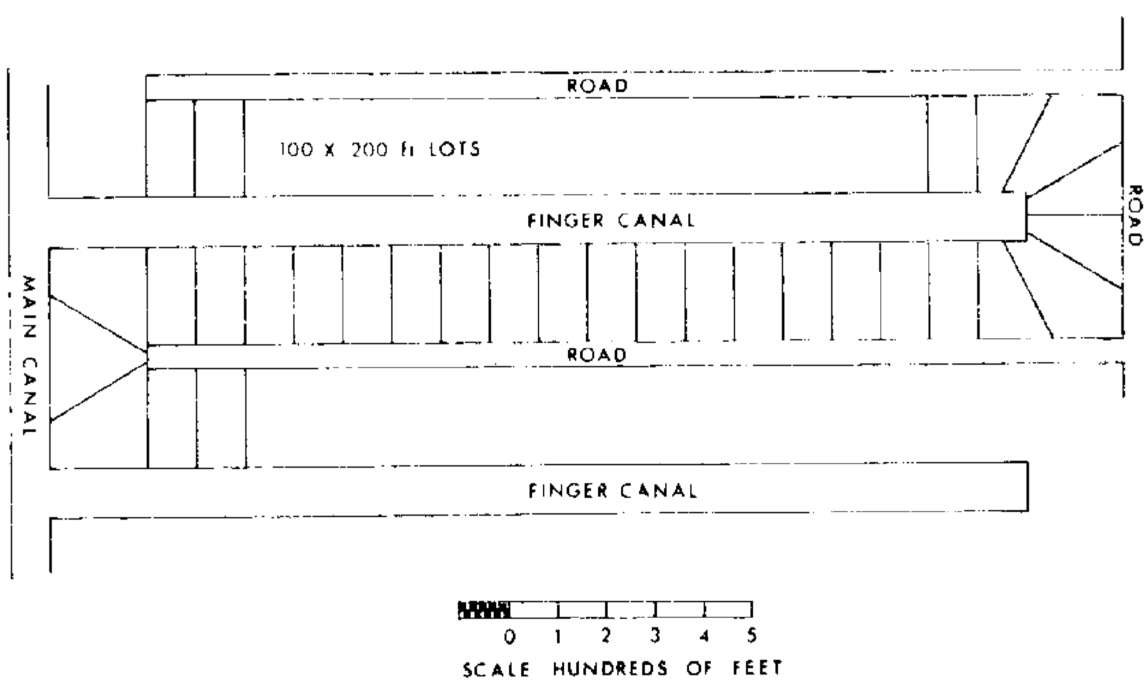


Figure 11.4 - Typical Residential Finger Canal Arrangement.

Table 11.3 - Dimensions and Parameter Values for Canal Design Element Simulations.

Finger canals:	
Length, ft	2000
Bottom width, ft	50
Inverse bank slope (dimensionless)	3
Mean depth, ft	8
Lot frontage, ft	100
Lot depth, ft	200
Distance between centerlines, ft	550
Alignment angle, degrees	270
Equivalent sand roughness, ft	5
Length of sections (uniform), ft	100
Tidal amplitude, ft	1
Tidal period, hr	12.42
Coefficients:	
Dimensionless longitudinal dispersion, K_x, sq ft/sec	0.1
Dimensionless lateral diffusion, K_y, sq ft/sec	0.1
Dimensionless vertical diffusion, K_z, sq ft/sec	0.0001
Background diffusion, E_0, sq ft/sec	0.0005
Vertical momentum transfer, N_z, sq ft/sec	0.002
Tidal entrance time decay, hr	1.0
Concentration:	
Pollutant inflow concentration, ppm	100.0
Pollutant inflow rate, ft^3/ft-hr	0.04
Background of receiving waters, ppm	0.00
Model:	
No. of lateral layers	1
No. of vertical layers	3
Time increment, DTH, hours (typical)	0.09703
No. of time steps/tidal cycle	128
No. of time steps between outputs (NPRINT)	1280
Total no. of time increments (NDT)	6400
Wind:	
Speed, mph	0 and 5
Direction from, degrees	090 and 270

11.2.3 Sensitivity of canal design element to wind. Consider first canal system A, a simple comb-structured design consisting of a primary channel and four finger canals (Figure 11.5). Canal reaches are designated by "R", and junctions by "J". All of the dead-ends are identified as J1, and the tidal entrance is J6. All canal reaches in the channel are joined at junctions, in which the model provides conservation of mass but does not include any diffusion. The surface area of a junction is relatively small compared with the surface area of a reach, and, therefore, there is relatively little error induced by ignoring dispersion in the junctions.

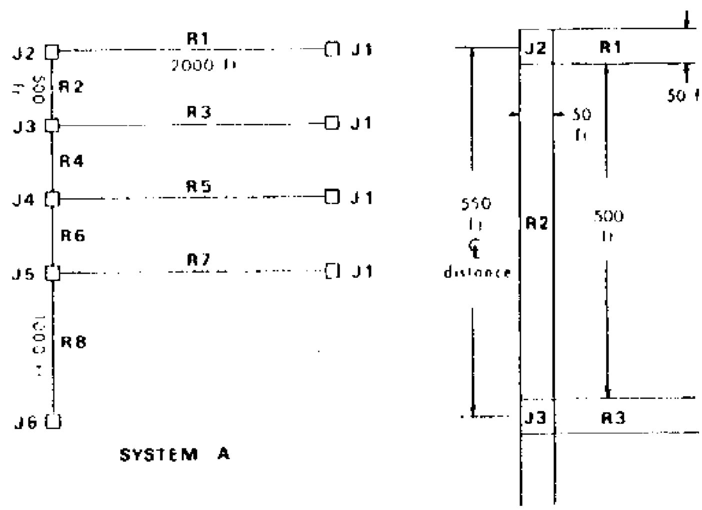


Figure 11.5 - Reach and Junction Numbers.

Under no-wind conditions, System A has not reached equilibrium after 30 tidal cycles. However, several characteristics are apparent which would not change with increased simulation time. Figure 11.6 shows surface layer and bottom layer concentration differences ($c - c_{RW}$, or c as in these tests the background concentration was set at zero) at the dead-ends of the finger canals, designated "Reach Dead-End, 1, 3, 5 and 7" and at the interior junctions J2, J3, J4, and J5. The surface concentrations are somewhat higher than the bottom concentrations at the two farthest junctions from the tidal entrance, since the pollutant has been introduced into the surface layer only. The amount of vertical mixing is essentially controlled by the vertical diffusion and vertical momentum transfer coefficients. As expected, the increased tidal velocities closer to the entrance induce greater vertical mixing, and there is little concentration difference between surface and bottom layers at these locations.

The surface concentration values at each of the four dead-ends of the finger canals are the same, and the bottom concentration values at each are the same. This result was also expected since, with the horizontal water surface assumption, the velocities are determined solely by the change in upstream tidal volume, and are, therefore, the same in each dead-end reach at the same distance away from the dead-end.

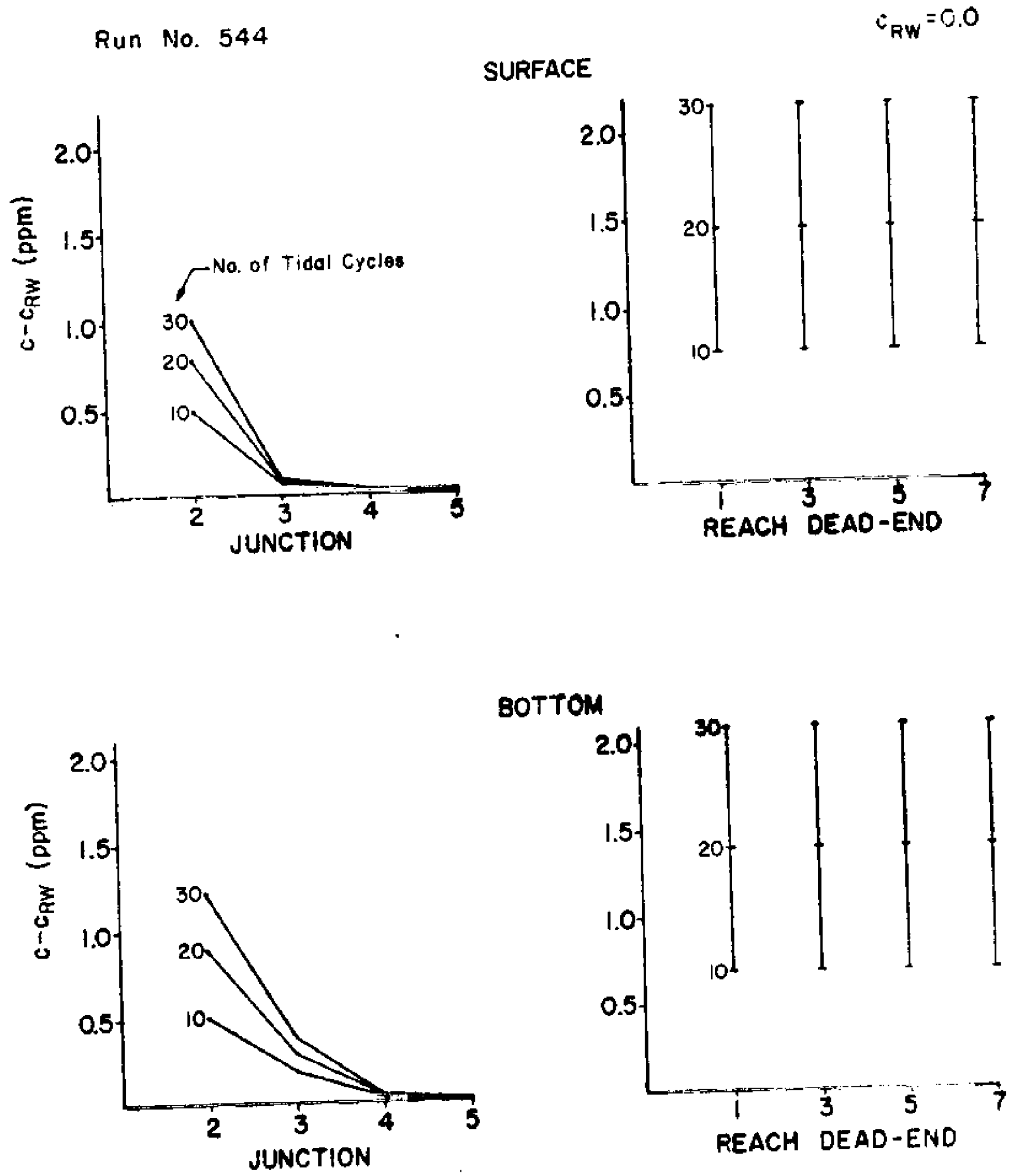


Figure 11.6 - Values of Surface and Bottom Concentrations for No Wind at Junctions and Dead-Ends at High Tide After Thirty Tidal Cycles, System A.

The superposition of a constant 5 mph wind blowing directly into the finger canals (a west wind, in this case) gives the results shown in Figure 11.7. In this case a background concentration, c_{RW}, of 5.0 ppm was used and only 30 tidal cycles were simulated. Comparing Figure 11.7 and 11.6, it can be seen that the surface and bottom concentrations after 30 tidal cycles are slightly (about 10 percent) lower at the dead-end of reach R1, and substantially lower at the dead-end of the other three reaches. This illustrates that even if the wind were blowing towards the dead-end, substantial flushing in all but the most remote locations from the tidal entrance would occur.

The results obtained when the direction of the wind was changed to the east are shown in Figure 11.8. There is less pollutant in the canal in this case, substantially less than in the two previous cases. The wind develops a high velocity away from the dead-ends in the surface layer, where the pollutant is being introudced.

In Figure 11.9 the difference in surface and bottom concentration profiles along the reach farthest from the tidal entrance, after 30 tidal cycles, for no wind, an upstream wind, and a downstream wind are compared. Again, it will be noted that better flushing is obtained for either wind direction than for no wind, which is due to the wind-induced vertical circulation.

11.2.4 Sensitivity of canal design element with basin. In order to evaluate the effect of additional tidal prism caused by the addition of a basin at the dead-end of a finger canal, a basin was added to System A, as shown in Figure 11.10, and the new system was designated System B. The dimensions of System B are the same as in System A except for the addition of the variable volume of water at the dead-end of the "northern" reach. The surface area of the basin has been arbitrarily set at 100,000 ft^2, about ten times the surface area of the canal network. As will be seen in the results of the simulation, this is unnecessarily large, leading to more than adequate flushing of all parts of the network except the inner halves of the finger canals. The basin acts as a source and sink of water for the system, so that a variable tidal prism can be used to tune the flushing of the network, but the model does not include any circulation or variation of concentration within the basin's boundaries. The layout and dimensions of System B are shown in Figure 11.10.

The tidal volume of the lake increases the tidal prism of the network, which effectively flushes the north reach (number R1) and the main channel to the tidal entrance (Figure 11.11). A direct comparison between this no-wind case and no-wind conditions for System A (Figure 11.6), considering that the latter was only simulated for 30 tidal cycles, shows that there is effectively no difference in concentration profiles at the dead-ends of the finger canals R3, R5 and R7. Again, these results stem from the fact that the tidal velocity in these reaches is a function of only the upstream tidal prism and distance from the dead-end. However, the concentration profiles for reach R3 in System B drop off toward very low values in the western half of the reach as pollutant mass is convected out of the system.

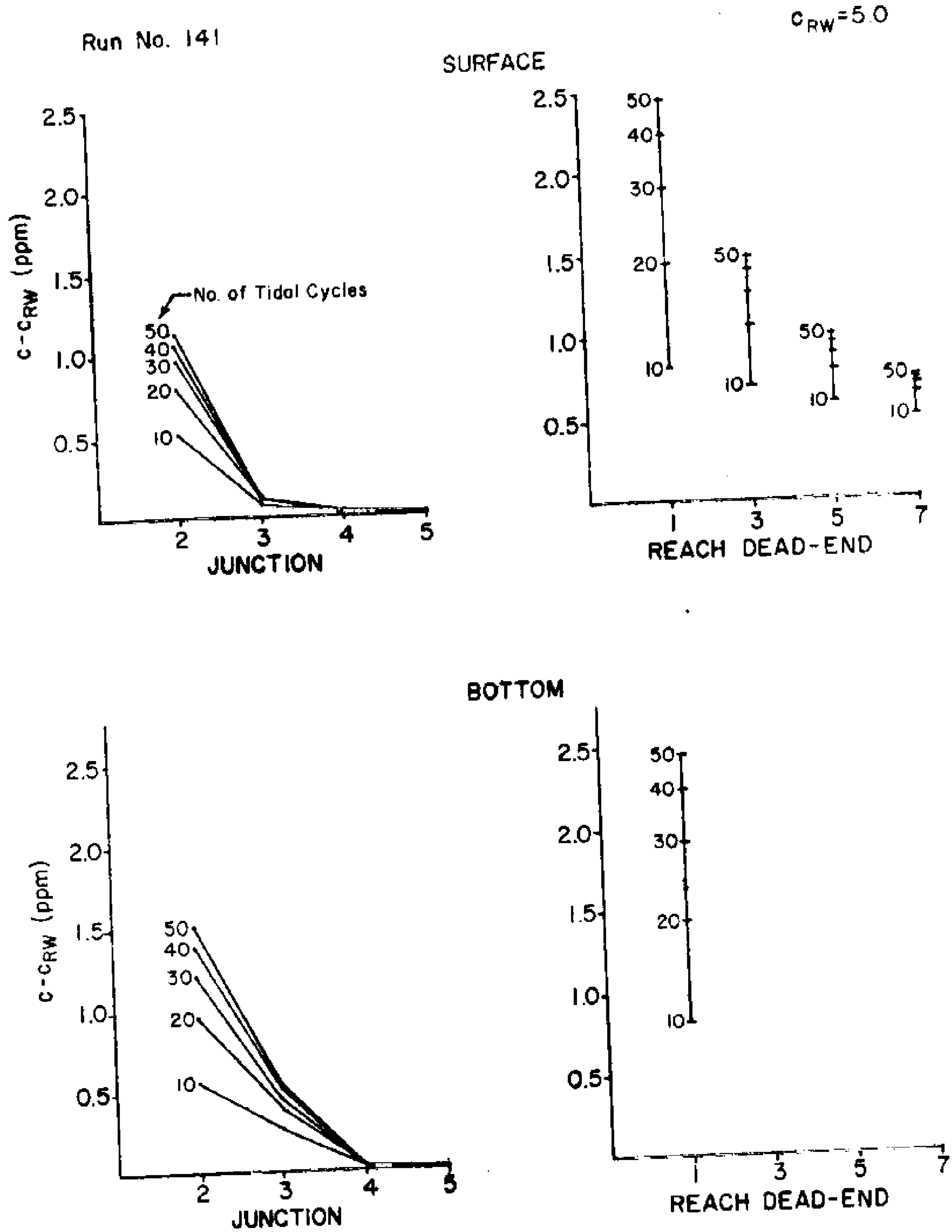


Figure 11.7 - Values of Surface and Bottom Concentration for West Wind, at Junctions and Dead-Ends at High Tide After Fifty Tidal Cycles, System A.

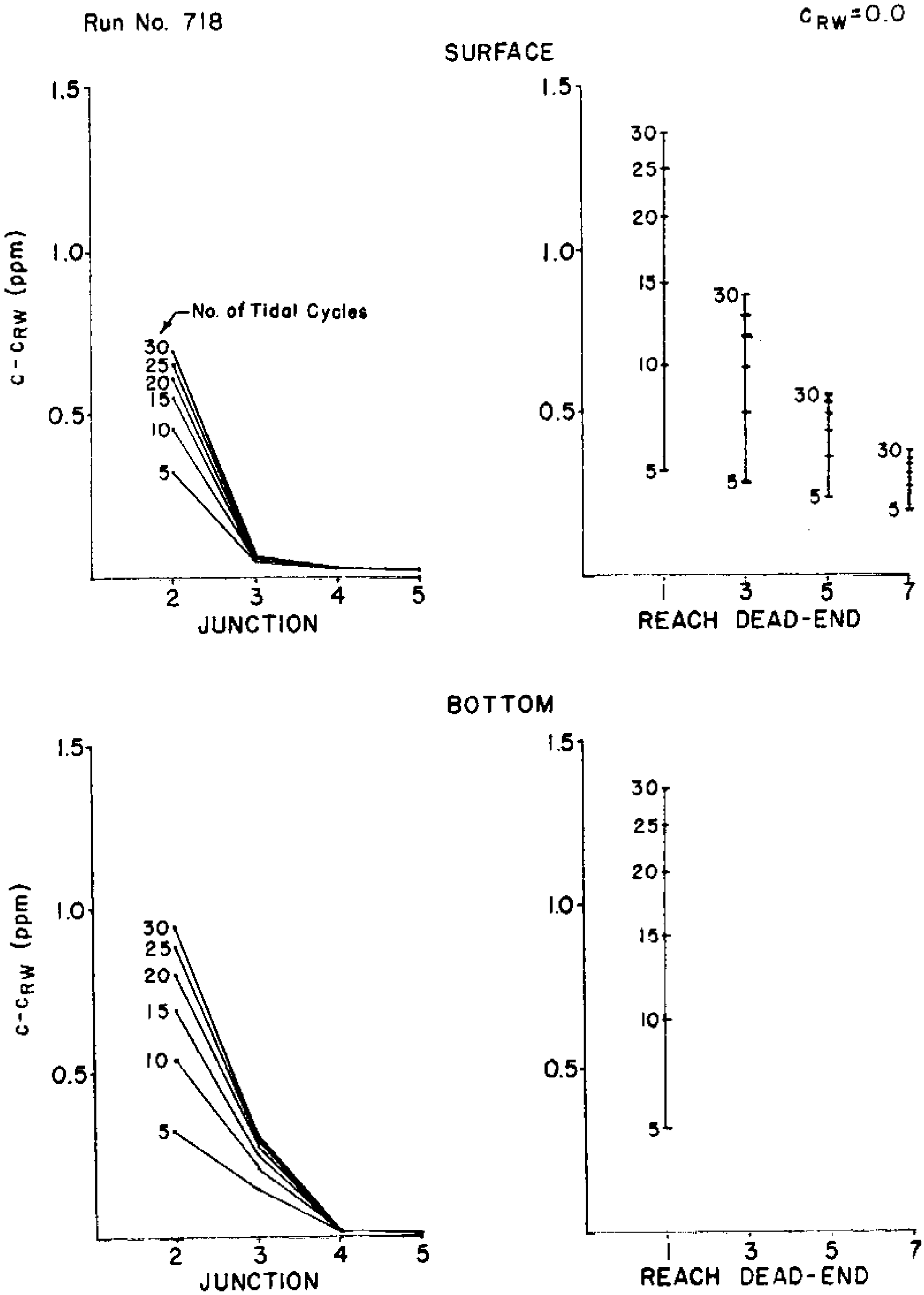


Figure 11.8 - Values of Surface and Bottom Concentration for East Wind, at Junctions and Dead-Ends at High Tide After Thirty Tidal Cycles, System A.

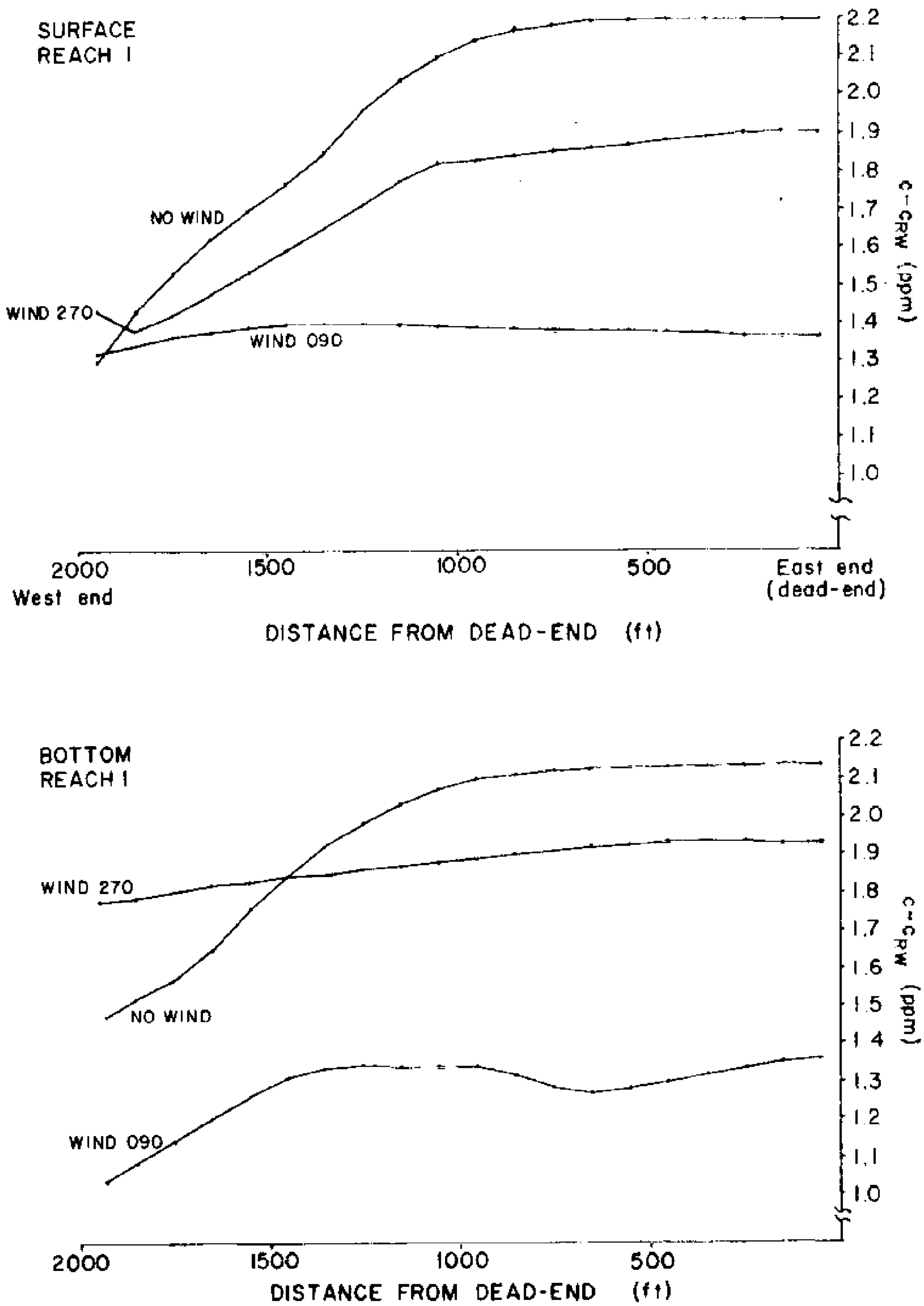


Figure 11.9 - Values of Surface and Bottom Concentrations for Three Wind Conditions in Reach Number 1 at High Tide After Thirty Tidal Cycles, System A.

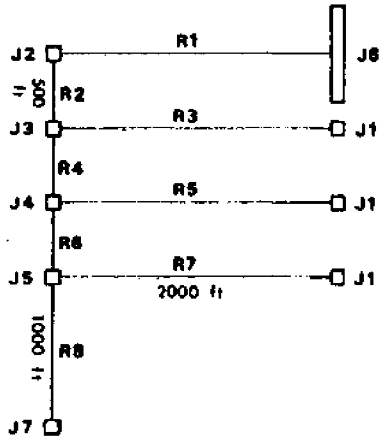


Figure 11.10 - Reach and Junction Numbers, System B.

The effect of superimposing a wind blowing into the finger canals is, as seen for System A, the production of a vertical circulation at the dead-end. This circulation carries pollutant mass into and out of the bottom layer, where it is relatively slowly convected out of the system. A comparison of the concentrations resulting from a west wind shows an order of magnitude decrease in concentrations in both surface and bottom layers at all junctions, and a decrease of about 75 percent at the dead-ends of the finger canals, over the no-wind concentration values. It was found that the concentrations are uniformly small throughout reach R3 for an upstream wind.

A wind directed toward the tidal entrance is even more effective in flushing pollutants from this system. Approximately the same pollutant concentrations occurred at the surfaces of junctions, but an order of 80 percent lower concentrations were found in the bottom layer and an order of 50 percent lower concentrations were found at the dead-ends of the finger canals, as compared with the effects of a west wind. A comparison of the concentration profiles in reach R3 shows this to be true in the surface layer in the inner half of the reach, and throughout the bottom layer.

11.2.5 <u>Summary of observations on canal design elements</u>. Concentrations at junctions will increase with distance from the tidal entrance. Under no-wind conditions, surface concentrations will generally be higher than bottom concentrations because (in these models) pollutants were introduced into the surface layer and vertical mixing is relatively slow. Under no-wind conditions concentration profiles in finger canals of equal length will be the same at any distance from the tidal entrance due to the horizontal water surface assumption. Furthermore, the concentration values at the surface will be the same, and at the bottom will be the same, for no-wind conditions.

A constant upstream wind into a finger canal transports higher-concentration surface waters to the dead-end, vertically downward, and into the middle and lower layers, providing some vertical mixing and generally lower concentrations. Floating material on the surface, however, would not be mixed by the same mechanism and would tend to collect at the dead-end. Convection of pollutants out along the bottom layer occurs under these conditions, but flushing is slow. The flushing of the main (in this case, the north/south) canal is by tidal action in these simulations since there is no north/south wind component. Through conservation of mass, however, this canal is flushed indirectly by wind action in the finger canals.

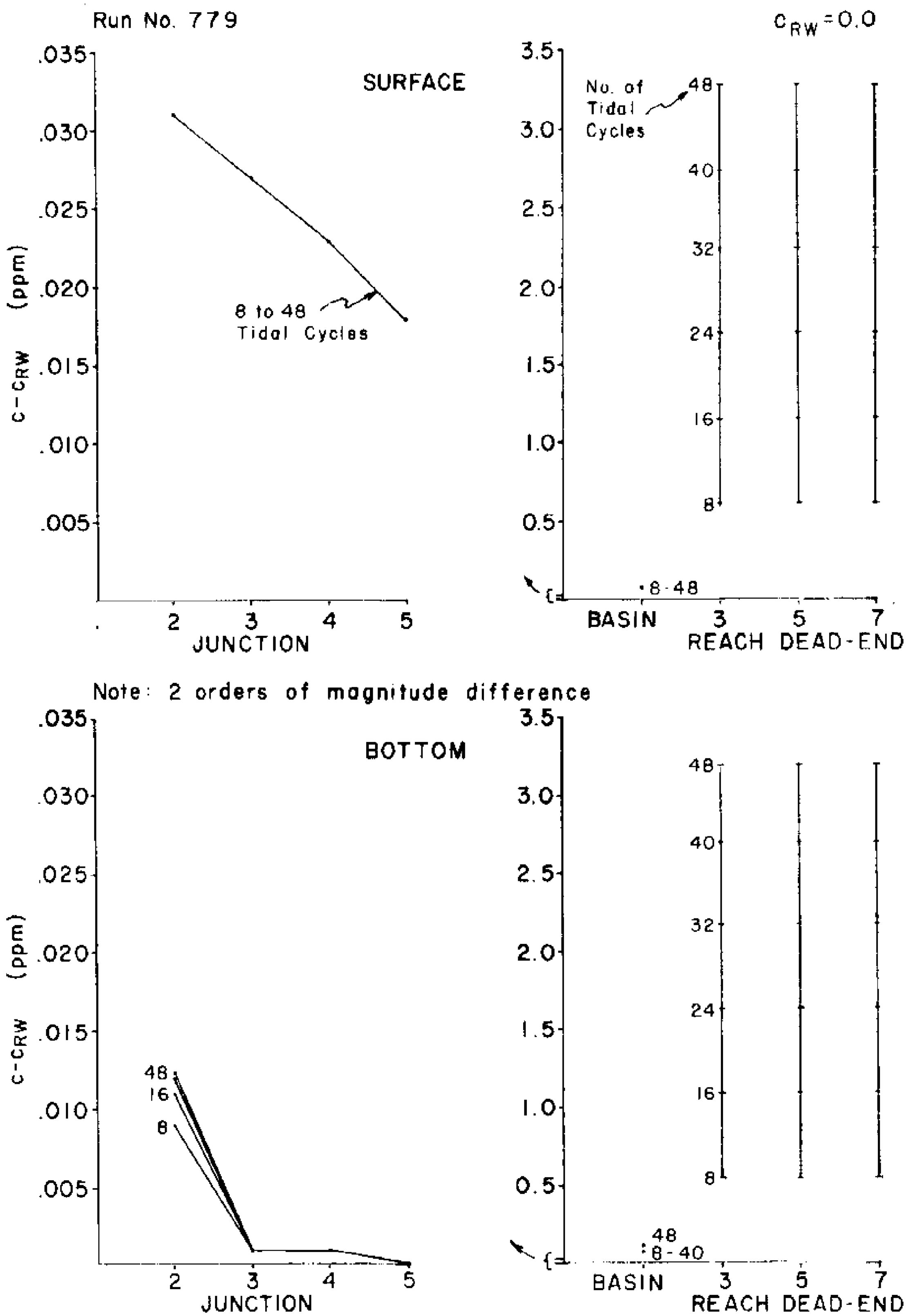


Figure 11.11 - Values of Surface and Bottom Concentrations for No Wind at Junctions and Dead-Ends at High Tide After Forty-Eight Tidal Cycles, System B.

A constant downstream wind provides the best flushing (considering only wind effects) at both junctions and dead-ends since it convects surface concentrations very effectively. On an incoming tide there is some transport of pollutants toward the dead-end, but the tidal flow will be relatively unpolluted water and upward flow at the dead-end will provide some vertical mixing.

The addition of even a small basin at a remote part of the canal network provides an effective method for increasing flushing in the reaches directly between the lake and the finger canals. It will also have some effect on flushing the outer portions of finger canals, due to increased convection, but it will have little effect on dead-ends.

CHAPTER 12
EVALUATION OF A CANAL NETWORK

The flushing effectiveness of a canal network should be evaluated by simulating the convection and diffusion that will occur under natural conditions. The natural variables that may be taken into account with the CANNET model are:

- topography of the site, in terms of elevations of the canal canal bed. Cross-sectional dimensions and layout of system.
- tidal elevation, either sinusoidal or discretized
- lateral inflow rate at any division and layer in the network
- concentration associated with lateral inflow
- first-order rate of decay of concentration in any division and layer in the network
- wind speed and direction.

The tidal elevation, lateral inflow and concentration, and wind may be changed at each time step in the simulation. The geometry of the canal and the concentration decay rate are fixed in time for any particular simulation.

12.1 Features of the Canal Model

A simple network showing the choices available for setting up a canal network in the canal model, CANNET, is illustrated in Figure 12.1. A canal *reach* is a length of a channel, or an entire channel, with uniform geometry. A reach may be divided into any number of equal or unequal length divisions, all with the same bed width, mean tidal depth, left inverse side slope, right inverse side slope, alignment angle, and equivalent sand roughness. If there is a change of geometry or roughness in a length of channel, then the channel should be divided into two or more reaches.

The *downstream* direction in any reach in the canal is defined in the direction from the point of the reach which is hydraulically farthest from the tidal entrance, along the centerline of the reach in the direction of the ebb tide. Reaches are connected by means of *junctions*. A junction can join from one to four reaches, designated the *right*, *upstream*, *left*, and *downstream* reaches proceeding clockwise around an observer facing downstream. A *loop* is defined as that part of a canal network in which a closed continuous line (the dashed line in Figure 12.1) can be drawn along the centerlines of component reaches.

A *basin* volume is connected at a dead-end. As mentioned previously, the basin adds a mass of water which increases the tidal prism of the system, but convection and diffusion are not modeled inside the basin.

A *bend* may be included in either the two- or three-dimensional version of the model (Figure 12.1). In the former helical flow is not modeled, and the only difference between a straight reach and a reach with a two-dimensional bend is that the component of wind acting along the axis of the canal in the bend is changed. The three-dimensional version of the model, CANNET3D, is required for an accurate assessment of the increase in flushing due to bends.

12.2 Variable Inputs for the Canal Network Model

The tidal elevation at the entrance can either be generated automatically as a sinusoid with the specified tidal amplitude and period, or can be read from a list of measured elevations during the simulation. Usually comparisons based on the sinusoidal tide are adequate for design purposes.

The lateral inflow may be either a constant flow with a constant concentration throughout a particular simulation, or specified for each time step. Furthermore, any computational cell in the model, which is designated by a particular division in a reach and by a particular layer, may have a lateral inflow which is different from the lateral inflow into any other cell (Figure 12.2). The most often used configurations, however, are either a constant lateral inflow along a reach, or a constant source at one cell in a reach. An inflow may also be turned on or off at any prescribed time during a simulation.

For some tests, it is useful to be able to set all of the cells in the network at the same initial concentration. For example, this will be useful when the designer is attempting to determine the relative effectiveness of flushing at different locations in the canal network.

The wind may be specified as constant in speed and direction over the duration of a simulation. Alternatively the wind may be varied from one time step to another, or may be specified at multiples of one time step with an automatic interpolation provided by the model between steps.

12.3 Trial Canal Design

A trial canal design is first formulated on the basis of the objectives agreed upon by the owner and the canal designer, and the results of the preliminary site investigation and the initial field surveys. At this stage, the designer will have preapred a detailed topographic map of the site and the surrounding area. This map should have been annotated with drainage patterns, the locations of areas of special concern, and the locations of existing canals and waterways. Then, following a set of design guidelines, the canal designer will develop a set of quantified design constraints and design criteria which are used in the design of the trial canal network. These steps are summarized in Figure 12.3.

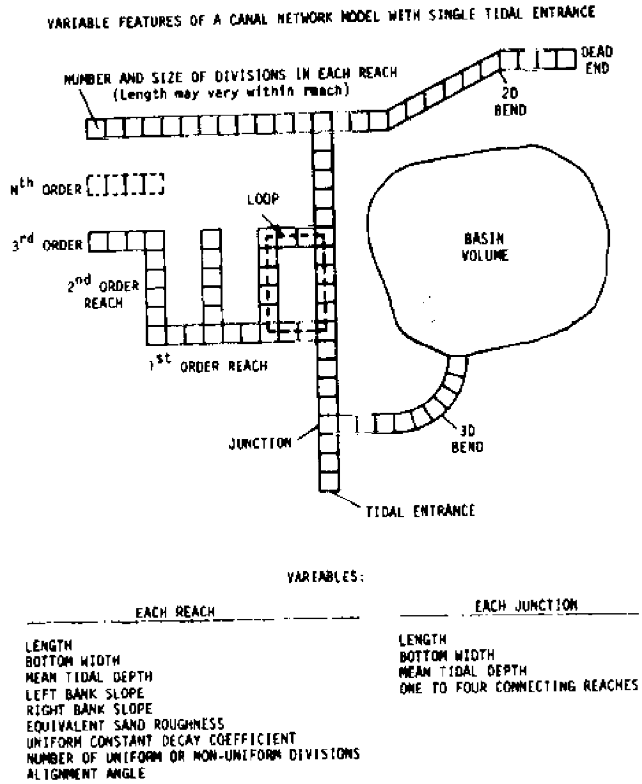


Figure 12.1 - Variable Geometrical Features of a Canal Network Model With Single Tidal Entrance.

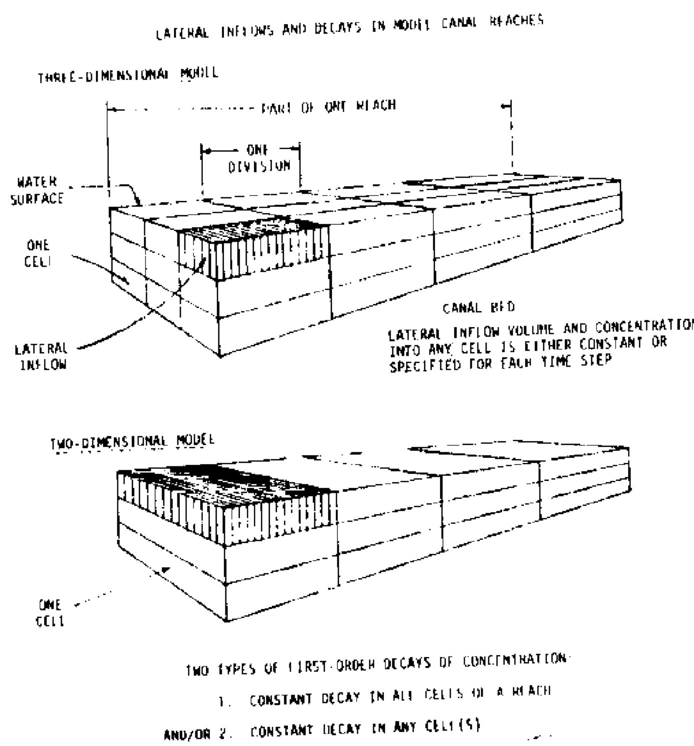


Figure 12.2 - Lateral Inflows and Decays in Model Canal Reaches.

For design purposes, a canal network may be visualized as consisting of a series of design elements, such as tidal entrances, lagoons, straight and curved channels, vegetated shallows, etc. Each of these design elements has certain features, both qualitative and quantitative, which are characteristic of that element. For example, a straight canal reach is characterized by its physical dimensions, the sizes and types(s) of materials which compose its bed and banks, its exposure to the wind and other features. The characteristics of these elements can be generalized and quantified to an extent, and therefore such elements can be assembled in various ways to create a canal system. Once assembled, the elements can then be sized to tune the system so that it meets the objectives established for it.

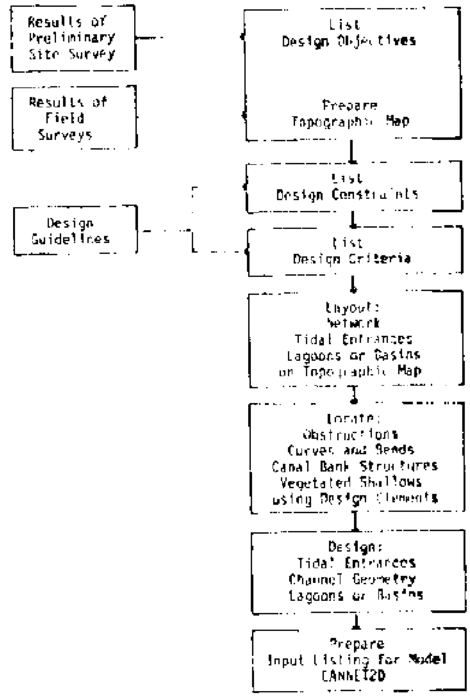


Figure 12.3 - Steps in Formulating a Trial Canal Design.

Since a site is a dynamic natural system with complex interrelationships among its many parts, the design should be guided by the fundamental principle that some, if not all, of its features will have a certain natural value to man. Whenever possible, an existing natural system should not be changed unless the overall value of the system will be increased and the additional cost of that increase is acceptable to the developer. The term "cost" is used here in its general sense, including the often nebulous environmental cost, and includes the long-term expense to the community as well as the short-term cost involved in the construction and the cost of marketing the product. In general, it is more expensive (i.e., it requires additional energy) to maintain a system which cannot maintain itself in relationship to the other natural systems with which it interacts.

12.4 Evaluation of a Canal Network

Once a trial canal design has been prepared, the evaluation procedure involves a number of simulations to establish the relative effect of changes in the design variables. To illustrate some of the considerations involved, the results of a succession of simulations of flushing in a *simplified*, hypothetical canal network are discussed in this section. The simplified nature of the canal network is described by completely uniform, rectangular geometry. This assures that comparisons of concentration distributions in various parts of the network may be more easily made.

It should be understood that the example developed here is for illustrative purposes only and is, therefore, somewhat artificial. Its purpose is to outline evaluation procedures. The resulting canal network is not to be considered in any way as a standard for good development practice.

Figure 12.4 is a diagram of the "existing" canal system which has been developed for this example. The site has 5000 feet of tidal shoreline and a depth of 5000 feet to the main road; an area of 573.9 acres. This design example is based on an assumption that only one tidal entrance can be used at the site, since this is the most common arrangement for residential canals along the southern and Gulf coasts. A small pond, six acres in surface area, lies near the main road, and water supply and sewage and stormwater connections are offsite.

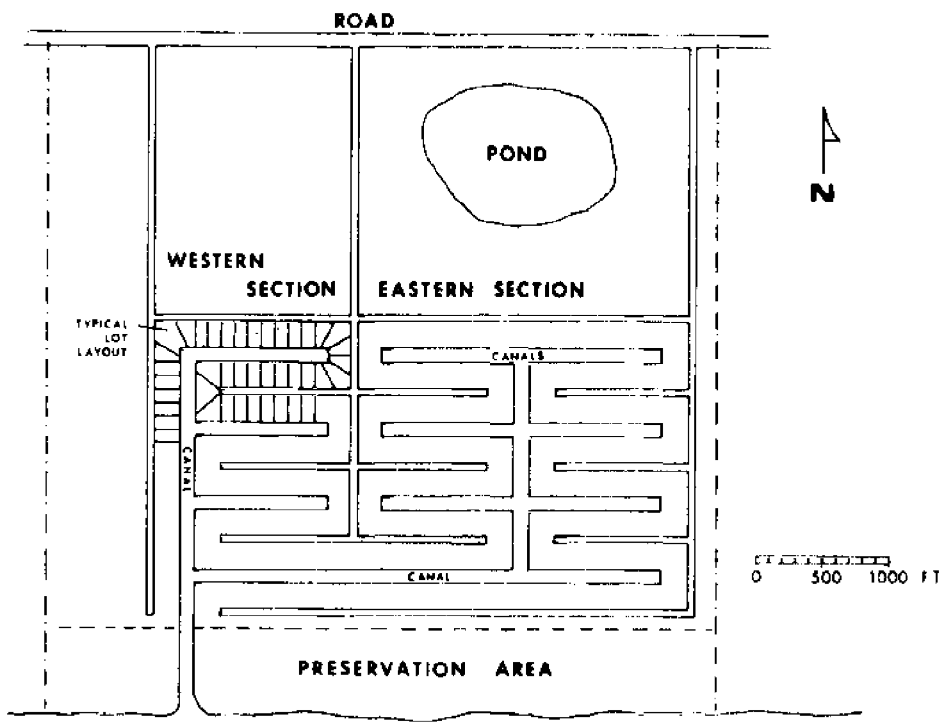


Figure 12.4 - "Existing" Example Canal System.

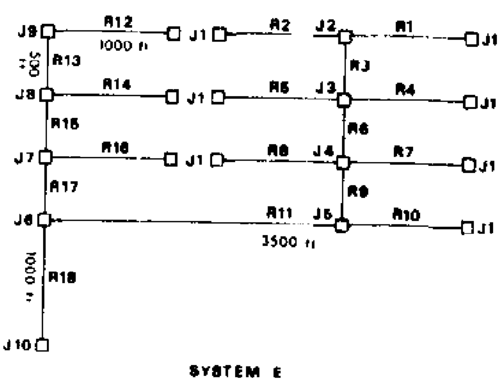
The walls of the 75-ft-wide canals in this example are vertical and bulkheaded along their entire length. Some of the bulkheads have shown a tendency to bow out into the canals, and objectionable odors from the canals and collection of trash at the downwind ends of finger canals have been experienced. The owner has decided, since there is space available along the canal frontage to provide sloping, vegetated banks for stability, and filter mounds and swales leading to a retention basin, to modify the canal banks. The mean tidal amplitude is 1 ft and the prevailing wind direction during the summer is east.

For purposes of comparing the effects of design modifications on the canal system, it is convenient to label the single set of finger canals the "western" section and the double set of finger canals the "eastern" section. The cross-sections of the canals are trapezoidal, with the same dimensions as used in the network sensitivity tests in Section 11.2.2 except that the length of all finger canals is 1000 feet instead of 2000 feet.

12.4.1 The example canal network model. In the interests of economy, the two-dimensional model CANNET2D should always be used unless there is a particular need to study a special feature that may have a critical effect on the performance of the network provided only by the three-dimensional version. Bends can be evaluated in two-dimensions by approximation with a few straight line segments, which ignores the additional vertical mixing induced by helical flows and provides a slightly conservative evaluation of a given network. The three-dimensional aspects of the network geometry, e.g., changes in depth or side slope, bridge crossings, etc., are handled routinely with the reach and junction structure of the two-dimensional model.

Prototype canal networks may have hundreds of finger canals and many miles of interconnecting channels. If a large system is to be designed, it may be desirable to replace portions of the network that can be isolated through a single branch canal with a basin or "lake" that has the same surface area as the replaced canals. The disadvantage of this procedure for simplifying a network is that circulation and concentration are not modeled through a junction, which is used only to conserve mass between adjoining reaches.

The layout of the example canal network (Figure 12.5) has purposely been made asymmetrical so that effects in these two parts of the network can be compared. For example, the comb-structure consisting of the three finger canals R12, R14, and R16 lying in the western portion is the same as System A discussed in Section 11.2.3 except the lengths of the finger canals have been halved to conserve computer time. The two facing comb-structures on the eastern side of the network, consisting of canals R1, R4, R7, and R2, R5, R8, were selected to show the effect on finger canals of both an upstream and a downstream orientation in conjunction with winds and with increases in the tidal prism. Provisions for adding a lake of variable volume at junction J2 at the north end of the eastern portion are included in the network. The model of the network is arranged with ten dead-end (J1) junctions, 8 interior junctions, J2 through J9, and eighteen reaches, R1 through R18 (Figure 12.5). When the lake and its associated reach are added, they are designated J10 and R19. The tidal entrance is labeled junction 10 without the lake, and junction 11 when the lake is included.



SYSTEM E

Note junctions are not to scale.

Figure 12.5 - Layout of Model Network, System E.

Results of all tests on the network are plotted on a common form to facilitate comparison (for example, see Figure 12.6). This form is arranged to scale, except that the area allocated for each finger canal is expanded to provide space for plotting concentration profiles. The vertically oriented graphs at the two sides of the form represent the concentration profiles in the two north-south canals consisting of reaches R13, R15, R17 and R3, R6, R9. Computed cross-sectionally averaged concentration, c_A, is plotted on a scale of from 1 to 10 ppm.

Each simulation began with a uniform concentration of 10.0 ppm at every cell in the computational network. The background concentration in the receiving water was set at 1.0. All simulations were run for fifty tidal cycles (621 hours, or over twenty-five days), and all plotted values of concentration are cross-sectionally averaged high tide values. Thus, the degree of flushing after fifty tidal cycles at any location in the network can be readily found from the network plots. The rate of flushing at increments of ten tidal cycles may be calculated from intermediate results in the computer printout.

A simulation of wind-induced flushing in the "existing" canal network is shown in Figure 12.6. This demonstrates that the system flushes relatively uniformly under a steady easterly wind of 5 mph.

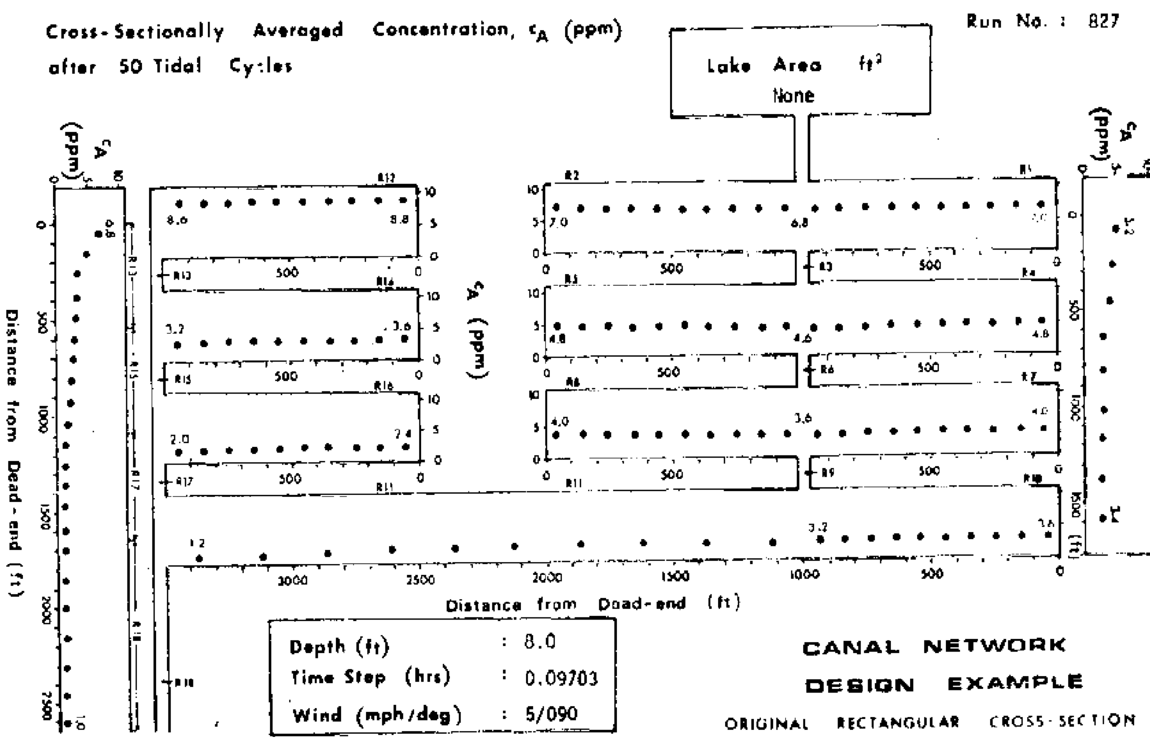


Figure 12.6 - Cross-Sectionally Averaged Concentration Distribution in the "Existing" Canal for 5 mph East Wind.

12.4.2 Simulation objectives. Before beginning the simulation of a trial canal design, some considerations should be given to the kinds of results which will be required for making design decisions. Specifically, the conditions which need to be simulated to provide reasonable assurance that the predictions of design behavior will be fulfilled when the prototype canal system has been constructed (or modified, if the network is already existing) must be defined. The following minimum steps will be required in using the model to prove a design:

1. Simulation of existing conditions in the canal system itself, if it already exists, or in a similar neighboring canal system, for calibrating diffusion coefficients by comparing computed results with field measurements;

2. Simulation of a different set of conditions in the same (or a similar) network for validating the settings in the model;

3. Sensitivity tests of the trial network design to determine desired network configuration and geometry;

4. Simulations with typical variable winds and tides for the site, for various pollutant inflow conditions, to define the operation of the network over the range of expected conditions at the site.

The network sensitivity tests, (Step 3 above), require the introduction of a substance into the canal network which can be used as an indicator of the effectiveness of flushing in various portions of the system. One procedure that has been found to be effective is to initially set the concentration in all cells in the model to a value of 10 ppm, and the background concentration to a value of 1 ppm. Then the concentration values after a number of trial cycles give directly the ratio c/c_{RW}, or the percent flushing that has occurred at a specific location in the network. The flushing times obtained by this process, while useful for making comparisons at different locations in the network, are unrealistically long because the rate of flushing is proportional to the concentration *gradients*, which take some simulation time to establish. A more realistic test condition, which will flush considerably faster, is a high point source concentration.

When the effects of variable winds and tides on a given network design are to be assessed (Step 4 above), several different procedures may have to be used. The flushing tests described for Step 3 above will still provide a measure of the relative effectiveness of flushing at any location in the network. However, realistic conditions of rates of inflow or outflow of a substance will also have to be simulated, which will require the definition of discharge, q_I, and concentration, c_I, for one or more cells in the model. The simplest of these types of "realistic" simulations to set-up is the introduction of a "point" source of pollutant in one cell, or several neighboring cells, at the beginning of the simulation. This is easily done by setting the initial value of the particular cell or cells to the concentration desired, and observing the rate of decrease of the resulting network concentrations as the substance is flushed under the variable effect of the wind. More complex simulations are possible, as, for example, the introduction of storm-water runoff into all the surface cells, or a time-varying septic tank discharge into the cells located at a particular section of a reach in the model.

12.4.3 Flushing under no-wind conditions. The proposed new trial canal design consists in substituting a trapezoidal canal cross-section for the original vertically bulkheaded cross-section in Figure 12.4.

Under no-wind conditions, this trial network flushes very poorly. Figure 12.7 shows that the tidal excursion is limited to approximately two-thirds of the distance into reaches R12, R14 and R16, and the length of reach R11 up to junction J5. Flushing is progressively better in the western section the closer the canals are to the tidal entrance.

12.4.4 Flushing with wind. When a moderate, steady wind is applied to the model canals, flushing is improved. The degree of improvement increases significantly with the wind speed. For a slight wind of only 2 mph from the east in the 8-ft-deep network, the effect on the eastern portion is negligible, but in the western part the downstream wind reduces the concentration at the dead-end of the northernmost finger canal (R12) from 10 to less than 7 ppm (Figure 14.6 in Morris, Walton & Christensen, 1978). When the wind is increased to a steady 5 mph, the concentrations at the dead-ends in the eastern portion decrease to approximately a third or less of their starting values, while in reach R12 the dead-end concentration decreases by one-half (Figure 12.8). In the case with 2 mph wind, flushing is still predominantly due to tidal action, while the higher wind (5 mph) has completely changed the concentration pattern to a comparatively uniform distribution.

Compared with the results for the vertically bulkheaded original design (Figure 12.6), concentration values in the finger canals are about 50 percent lower in the 8-ft-deep trapezoidal canal design. This is due primarily to the effect of wind on the increased surface area.

In the simulations considered thus far, the vertically-averaged values of concentration at the dead-ends of reaches R1, R2 and R12 decay the least rapidly of any location in the canal network. When the average concentrations at a given location for a particular simulation are plotted versus time, as for example in Figure 12.9, they often closely fit a first-order decay characteristic after ten or twenty tidal cycles. If it is assumed that the decrease in concentration will continue to follow the first order decay relationship, given by

$$\frac{c}{c_0} = K_0 e^{Kt} \tag{12.1}$$

where c_0 = reference concentration, [dimensionless]; K_0 = constant associated with an initial value, [dimensionless]; e = the exponential constant; and K = decay coefficient, [1/T], the flushing time to 10 percent of the initial value may be found from any two points on the concentration plot, either

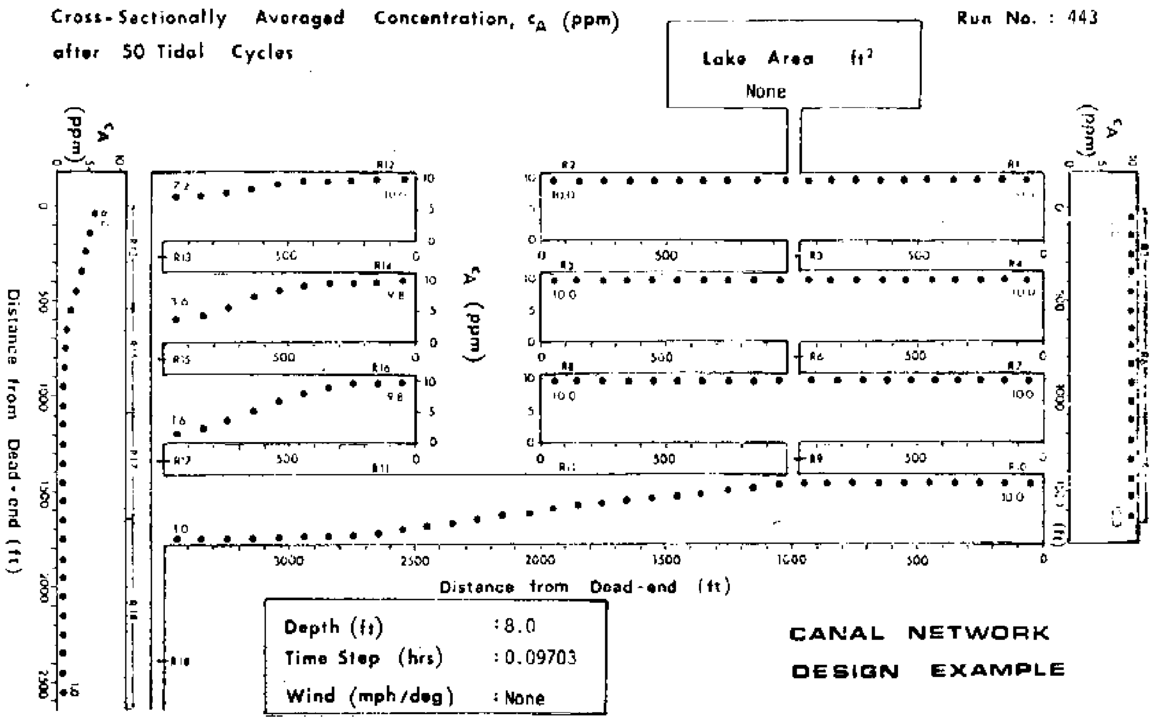


Figure 12.7 - Cross-Sectionally Averaged Concentration Distribution in Bottom Layer of System E, 8-ft-deep Version with no Wind.

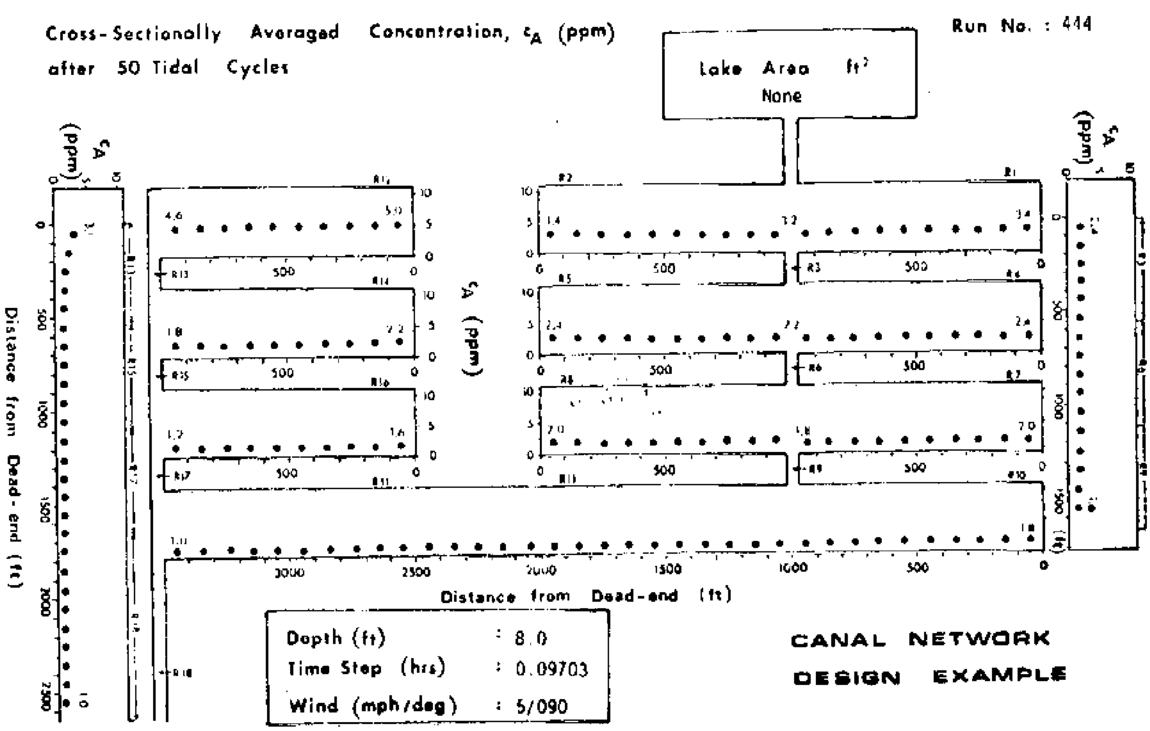


Figure 12.8 - Cross-Sectionally Averaged Concentration Distribution in Bottom Layer of System E, 8-ft-deep Version with a Steady East Wind of 5 mph.

measured or best fit. Designating the two points (c_1, N_1) and (c_2, N_2), where N_1 is less than N_2, N represents the number of tidal cycles since the start of the simulation and T is the tidal period.

$$K = - \frac{\ln(c_1/c_2)}{(N_2 - N_1)T} \qquad (12.2)$$

and

$$N_{10\%} = (N_2 - N_1) \frac{2.3}{(c_1/c_2)} \qquad (12.3)$$

where $N_{10\%}$ = number of tidal cycles to reach 10 percent of initial concentration.

Figure 12.9 shows the decrease in *bottom* concentration at the dead-ends of canals R2 and R12 in the 8-ft-deep network with a 5 mph east wind. The points at thirty through fifty tidal cycles lie on a straight line on the semi-logarithmic coordinates. Thus, if the assumption is valid that the decrease in bottom concentration follows a first-order decay relationship, these lines may be easily extrapolated by using equation (12.3) to 10 percent. The resulting flushing times are 76 tidal cycles for canal R2 and 126 tidal cycles for canal R12.

12.4.5 Flushing with additional tidal prism. When a small waterbody, such as a boat basin, is connected to the canal network at some distance from the tidal entrance, the additional tidal prism provides higher discharges and velocities in the canals comprising the principal flow path to the waterbody. In addition, pollutants convected through the mouth of a finger canal into the main canal on ebb tide are mixed with the greater tidal flow, resulting in lower concentrations to be transported back into the fingers on flood tide. Thus, it is expected that the addition of a waterbody or "lake" to the canal network will result in substantially lower concentrations in the main reaches R3, R6, R9, R11, R13, R15, R17 and R18, and some effect in the outer portions of the finger canals.

The surface area of the trial canal network is 1,680,000 ft^2. The surface areas of the lakes are 250,000 ft^2 (15 percent of total network area) and 500,000 ft^2. The results for the 8-ft-deep network, no wind, and the smaller lake area, are shown in Figure 12.10. The results of this simulation show that the concentration values in the main channels are reduced approximately 50 percent after fifty tidal cycles, and that the increased flushing extends, with decreasing effect, almost the entire distance into the finger canals. Comparison with Figure 12.7 shows substantial improvement in the eastern portion of the network.

When the surface area of the lake is increased to 30 percent of the area of the canal network (Figure 12.11), the concentration values in the main channels between the tidal entrance and the lake are reduced to about 15 percent of their initial values after fifty tidal cycles, with proportional

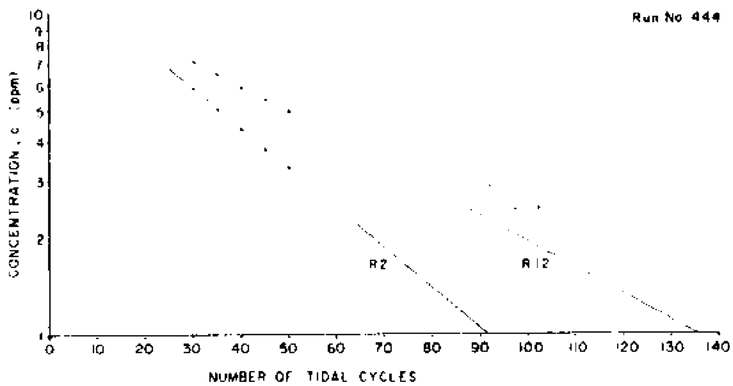


Figure 12.9 - Semi-logarithmic Plots of Laterally Averaged Bottom Concentrations at the Dead-ends of Canals R2 and R12 versus Number of Tidal Cycles, in the 8-ft-deep Version of System E, With a Steady East Wind of 5 mph.

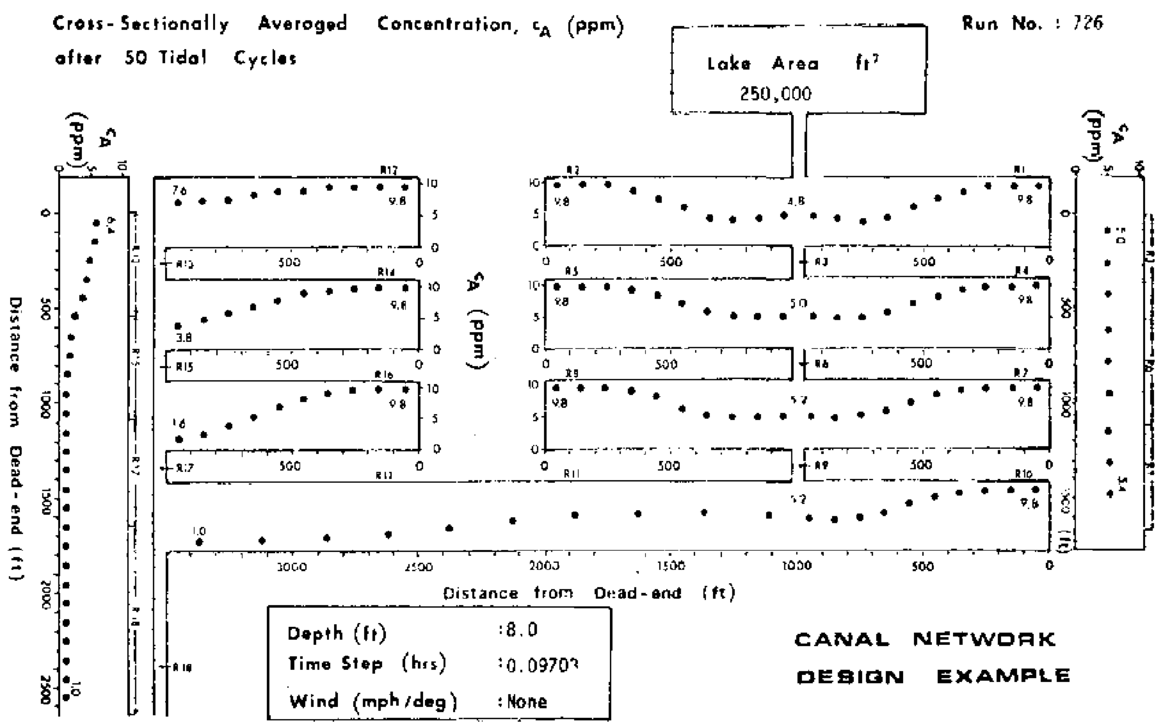


Figure 12.10 - Cross-Sectionally Averaged Concentration Distribution in Bottom Layer of System E, 8-ft-deep Version with no Wind and a Lake with a Surface Area of 15 Percent of Canal Network Surface Area.

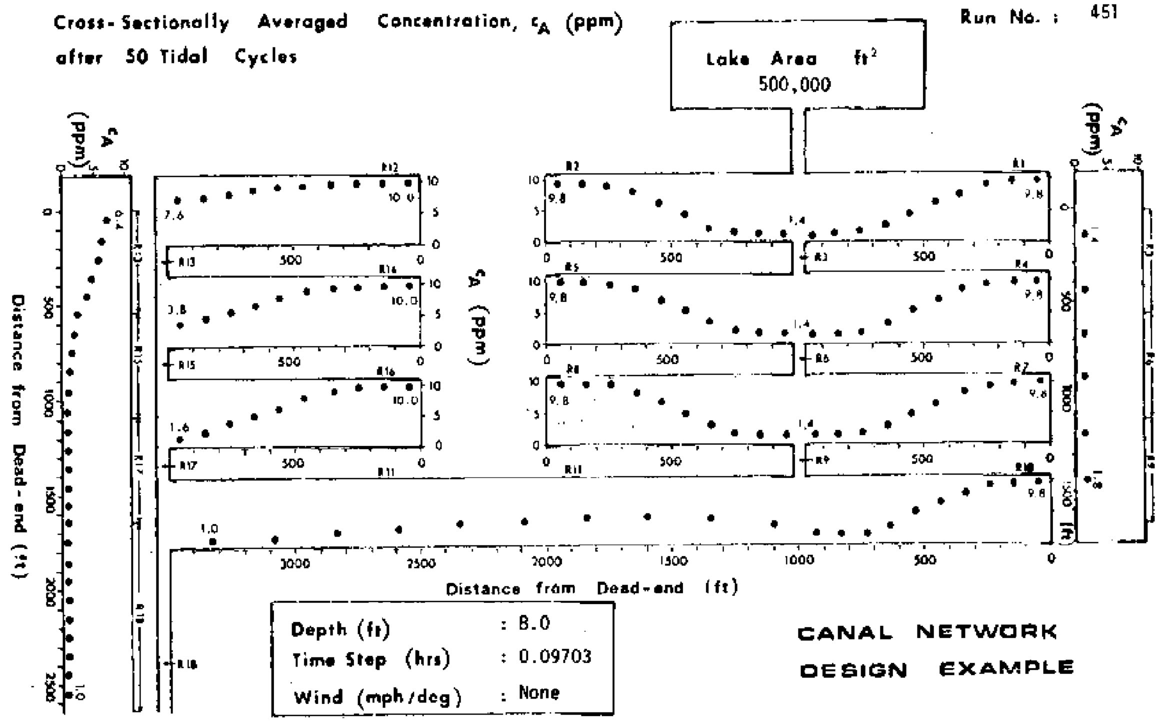


Figure 12.11 - Cross-Sectionally Averaged Concentration Distribution in Bottom Layer of System E, 8-ft-deep Version with no Wind and a Lake with a Surface Area of 30 percent of Canal Network Surface Area.

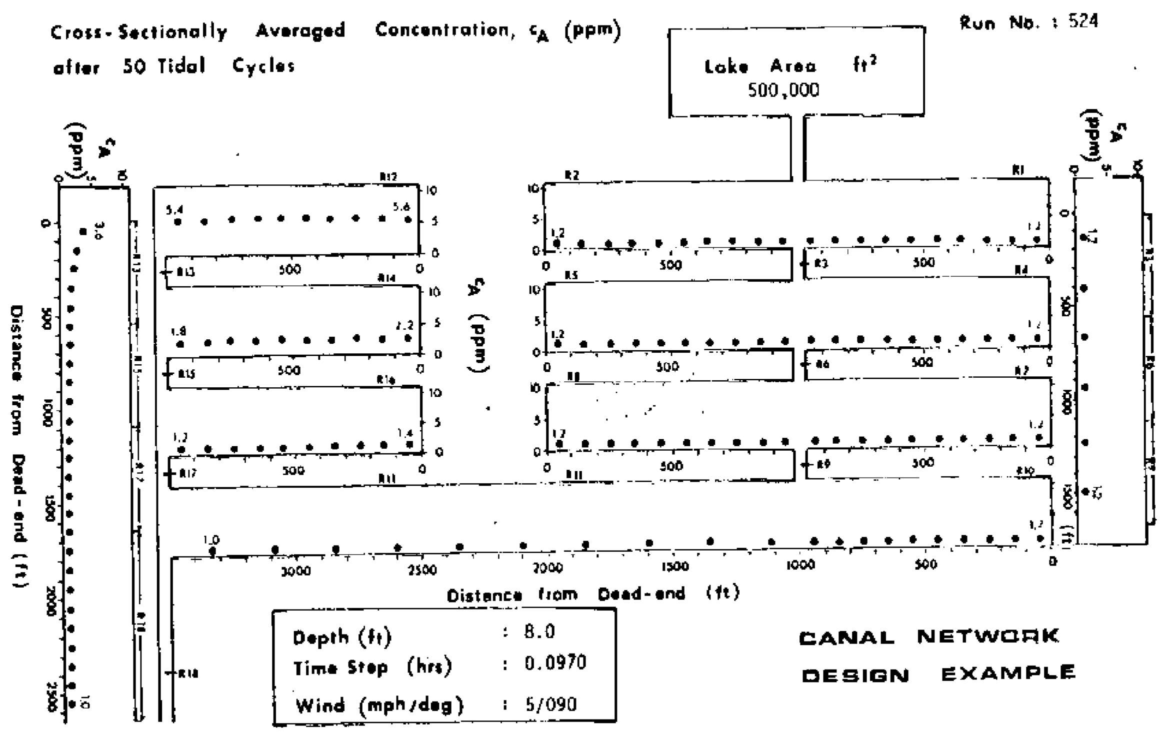


Figure 12.12 - Cross-Sectionally Averaged Concentration Distribution in Bottom Layer of System E, 8-ft-deep Version with East Wind of 5 mph and a Lake with Surface Area of 30 Percent of Canal Network Surface Area.

decreases in the eastern finger canals. No effect in either case is experienced in the western part of the canal network, and the effect is negligible at the dead-ends of the finger canals.

When combined with an east wind of 5 mph in the 8-ft-deep system, the concentration profiles in the eastern portion of the network are uniformly reduced to 12 percent of their initial values after fifty tidal cycles (Figure 12.12). Comparing Figure 12.12 with the results from identical conditions, except for no lake (Figure 12.8), shows an improvement from 34 percent to 12 percent flushing at the dead-end of canal R2, and a slight but negligible increase in concentration at the dead-end of canal R12. The combinaton of wind and extra tidal prism is obviously, from the simulations presented in this section, the most effective flushing mechanism presented thus far for tidal canal networks.

12.4.6 Comparison of effects of variable wind. From the comparative tests described so far, it is evident that the results of a given simulation depend more upon the wind than upon the tide. For the previous tests a steady wind and a harmonic tide have been applied to the model in order to simplify the analysis and comparison of results. For an actual design, it would be more appropriate to use a typical time-varying wind and tide to ensure more realistic results.

To explore this possibility, a variable wind sequence was used with the harmonic tides to obtain a network concentration distribution after fifty tidal cycles. This wind sequence was designed as an approximation to measured winds at a canal site south of Jupiter, Florida, by specifying a typical wind speed and direction at four times during the day and using the canal model, CANNET, to interpolate through each resulting three-hour period. The following wind sequence was used:

	Specified Wind	
Time	Speed, mph	Direction
0300	0	NW
0900	6	NNW
1500	8	N
2100	0	NE

Both speed and direction are interpolated in the model, so there will be a gradual shift in direction from northwest to north-northwest as the wind picks up in the early morning hours. The 12-hr period for no wind at nights is typical of October winds in the region south of Jupiter Inlet. If the network model described in this section were to be used to simulate a canal system at a site on the east coast of Florida, the tidal entrance would most likely be on the east side. Therefore, to provide a realistic simulation, the above winds were rotated ninety degrees clockwise to match the orientation of the reaches in the model. The simulation was arbitrarily started at the 0300 wind.

An "equivalent" steady wind, for purposes of comparison, was defined by noting that the shape of the wind velocity distribution over the 18-hr period during which the variable wind is active, is approximately Gaussian. Taking the peak value to be 8 mph, numerical integration at one hour intervals over 24 hours provided a time-mean velocity of 2.09 mph. Thus, a wind of 2 mph from North was used for the "equivalent" steady wind.

The results of an East wind of 2 mph were shown in Figure 14.6 in Morris, Walton & Christensen [1978]. The results from a simulation using the variable wind are shown in Figure 12.13. From a comparison of these two figures, it is obvious that the variable wind has provided a great deal more flushing than the "equivalent" steady wind. The most likely explanation is that, when periods of maximum tidal velocities occur simultaneously with intervals of high wind velocity (on the order of 6 to 8 mph), there is a great deal of mixing and movement of pollutant in the system. This activity, even over a relatively short interval of time, is far more effective in flushing the network than the steady 2 mph wind, which has been shown to be very limited.

It may be concluded from the foregoing comparison that the simulation of the transport of substances in a trial canal design should be conducted with variable wind data appropriate for the site. Unfortunately, even wind data from a nearby airport cannot be expected to correlate closely with actual conditions at the site. Since variable winds are important in their effect on the results obtainable from the model, and published data are not transferable to different sites, it follows that the canal designer should take wind measurements at the site over a sufficient period of time either to find a representative wind sequence or to cover the entire length of time to be used in the canal network simulations.

12.5 Evaluation of Mean Tidal Depth

An evaluation of the optimal mean tidal depth, or any other geometrical variable, may be accomplished in a more comprehensive manner by comparing the effectiveness of flushing the same total initial mass of a pollutant from each system. If an arbitrary pollution mass is selected to give a reasonable initial concentration for one canal volume, such as 10 ppm, then the initial concentration for any other canal geometry may be obtained from the relationship

$$c_{b\cdot i} = \frac{\forall_a}{\forall_b}\, c_{a\cdot i} \tag{12.4}$$

where $c_{a\cdot i}$ and $c_{b\cdot i}$ are the initial concentrations of pollutant in networks with volumes $\forall_a$ and $\forall_b$, respectively, and where the densities of the pollutant and the canal waters are assumed equal.

The comparison tests are run by setting all compuational cells in the model to the concentration calculated from equation (12.4). The resulting changes in concentration distribution with time are plotted, for these tests, is a pseudo-three-dimensional form. The plot is obtained by mapping the network with points at 100 ft increments along each channel and then rotating the map to a prescribed azimuth (in this case, 60°) and elevation (in this case, 25°). The concentration values, however, are all plotted at their true scale and are not adjusted for their location in the third dimension.

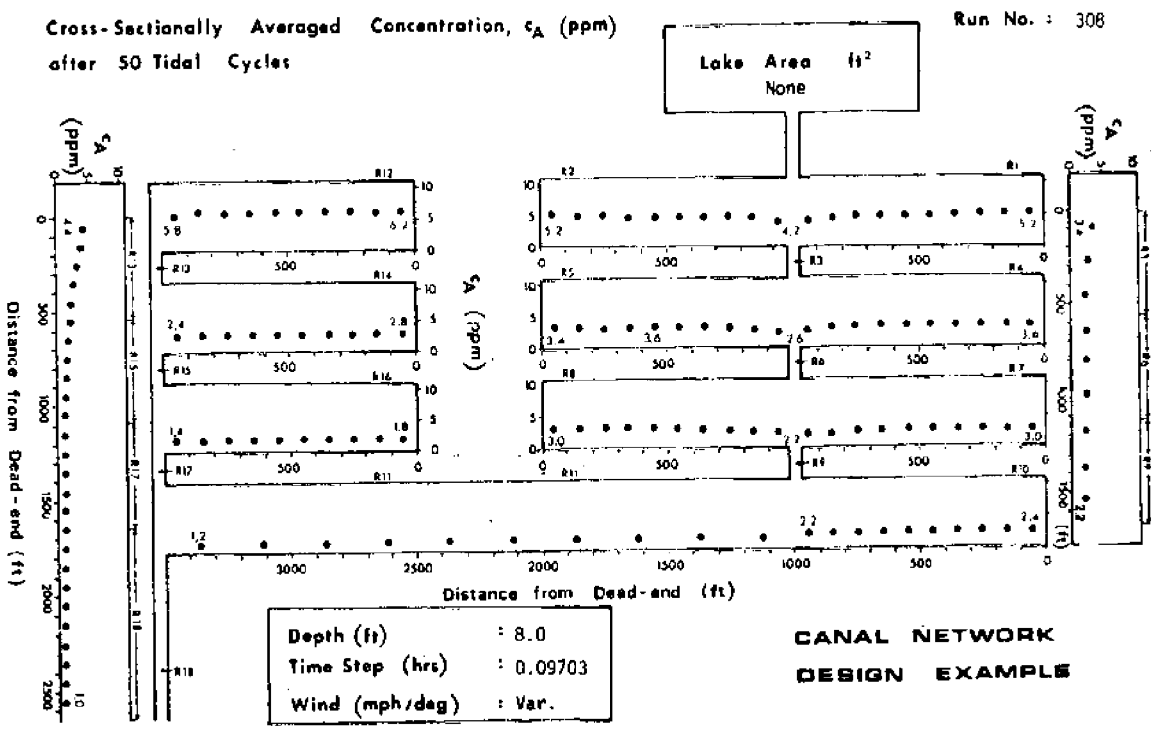


Figure 12.13 - Cross-Sectionally Averaged Concentration Distribution in Bottom Layer of System E, 8-ft-deep Version with a Variable Wind Speed and Direction and no Lake.

For evaluating the effects of changes in the mean tidal depth, each test was started with an initial uniform concentration of 10 ppm for 8 ft deep canals, 23.87 ppm for 4 ft deep canals and 5.74 ppm for 12 ft deep canals, at every cell in the computational network.

The results of a flushing simulation of System E with no wind, after 50 tidal cycles, are shown in Figure 12.14.

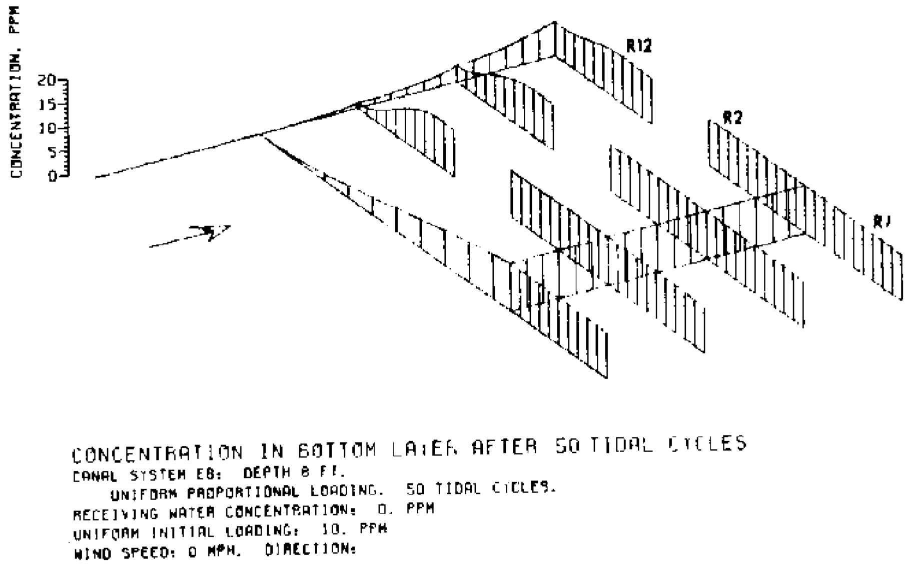


Figure 12.14 - Concentration Distribution in Bottom Layer of System E, 8-ft-deep Version, After 50 Tidal Cycles With No Wind.

The effective excursion distance of the tide under these conditions is seen to be limited to the two principal channels and approximately one-half of the distance into the finger canals in the western section of the network. A comparison of the differences between the values of concentration at the surface and at the bottom at the dead-ends of reaches R1, R2, and R12 shows these differences to be less than one percent.

As described previously, a wind will set-up a vertical circulation pattern that results in improved flushing in both the surface and bottom layers. It is to be expected that the greater the width of the channel, the greater will be the wind-induced flow in each layer. Since all channel dimensions in these simulations are defined with a constant bottom width (50 feet), the surface width will increase with depth and the effective wind-induced flushing should therefore increase with depth.

When a steady, moderate, 5 mph wind from the east is superimposed on the 8 ft deep network, flushing is substantially improved. The highest concentration (4.4 ppm) in the system (Figure 12.15) is found at the dead-end of Reach R12. Reversal of the wind to the west would, from the results observed with System A, result in a somewhat higher concentration in reaches R1 and R12. An increase in the wind speed to 10 mph decreases the concentrations at the dead-end of reach R2 from 2.6 to 1.1 ppm, but has relatively little effect on the concentration at the dead-end of Reach R12 (4.05 ppm for 10 mph wind) as shown in Figure 12.16.

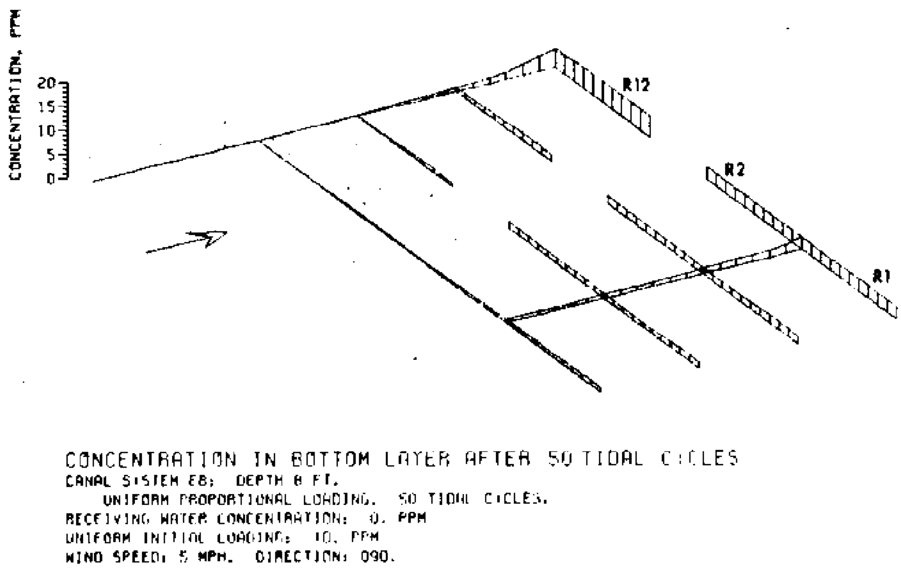


Figure 12.15 - Concentration Distribution in Bottom Layer of System E, 8-ft-deep Version, After 50 Tidal Cycles, With East Wind at 5 mph.

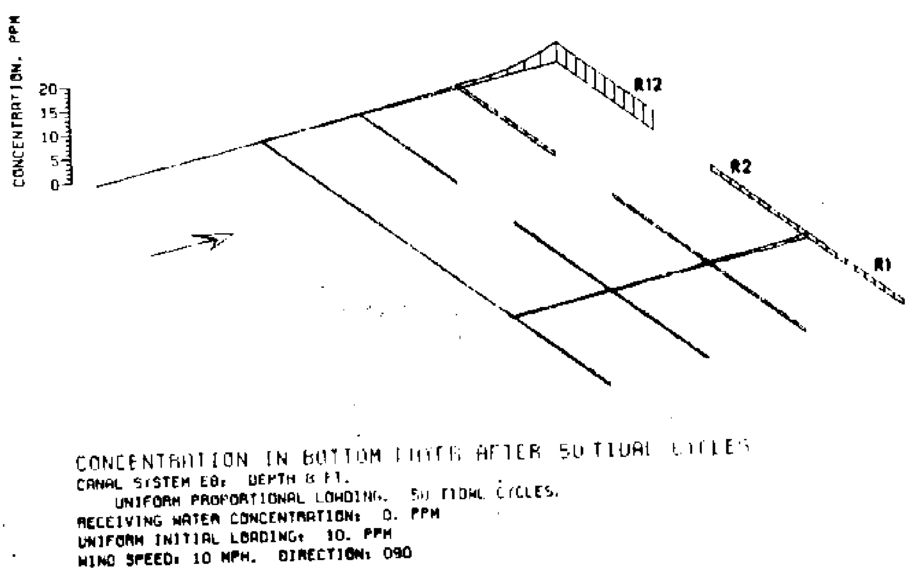


Figure 12.16 - Concentration Distribution in Bottom Layer of System E, 8-ft-deep Version, After 50 Tidal Cycles, With East Wind at 10 mph.

The effect of a change in the uniform depth of the canal network from 8 feet to 4 feet, for a constant easterly wind of 5 mph, is shown in Figure 12.17. It can be seen that the tide performs most of the flushing in this network, and that the wind is much less effective in this network than in the 8-ft-deep system. A plot of the results for the same conditions in a 12-ft-deep canal, compared with the results for the 4- and 8-ft-deep network, shows that the 12-ft-deep canal has the least flushing of the three in the eastern section, but not quite as good flushing as the 4-ft-deep canal in the western section (Figure 12.18). These results differ somewhat from those obtained for a 2000 ft straight canal, with wind directed into the canal (Figure 11.2). In that case, for winds of 4 mph and above, the flushing improved directly with depth.

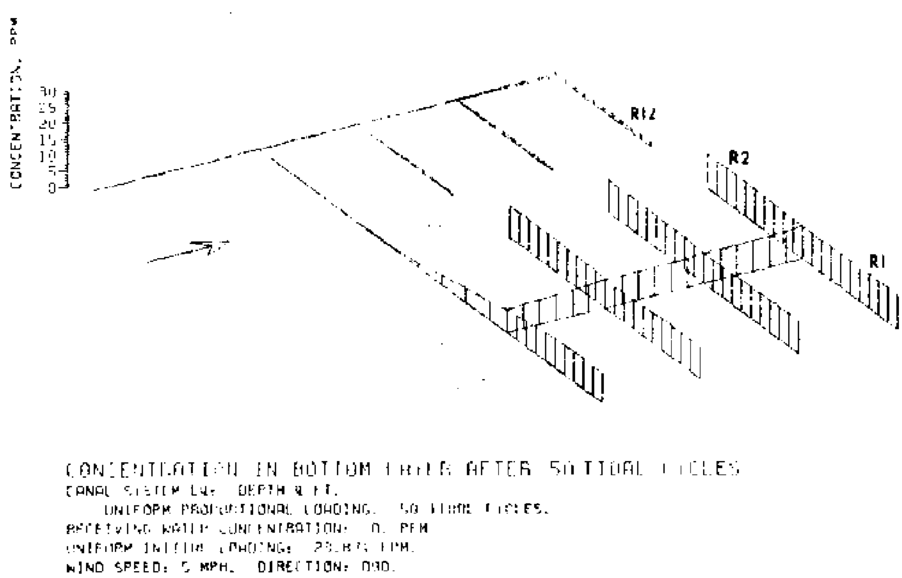


Figure 12.17 - Concentration Distribution in Bottom Layer of System E, 4-ft-deep Version, After 50 Tidal Cycles, With East Wind at 5 mph.

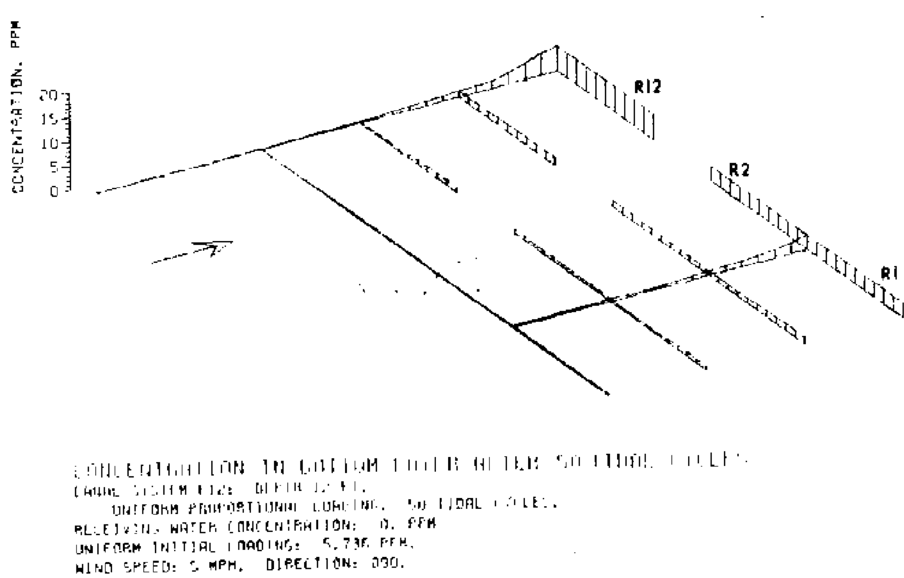


Figure 12.18 - Concentration Distribution in Bottom Layer of System E,12-ft-deep Version, After 50 Tidal cycles, With East Wind at 5 mph.

The effects of changes in uniform mean tidal depth and wind speed on the mean concentration at the dead-ends of Reaches R1 and R12, which are the locations in the network at which the slowest rates of flushing occur, are summarized in Figure 12.19a and b. It can be seen that, at least at these two dead-ends, flushing increases with wind speed. For very shallow depths a high constant wind speed would be required to obtain reasonable flushing, but as the depth is increased toward 12 feet, the value of wind speed becomes less important.

In reach R1 there would appear to be an optimal depth within the range of 8 to 10 feet, assuming a statistically equal likelihood of east wind velocities between zero and 10 mph. In reach R12, however, the optimal flushing occurs at the two extremes of tested depths.

Considering that Figure 12.19 provides comparisons only for constant easterly winds, and only at the two dead-ends of Reaches R1 and R12, a judgement cannot be made with these limited data as to the best depth. However, additional tests with variable winds, tabulated against the mean concentration in the entire network as well as in selected portions of the network, would greatly assist in such a determination.

12.6 Summary of Evaluation of Example Canal System

From the various kinds of tests and limited comparisons presented in this example, it can be seen that the potential for evaluating many different configurations of canal geometry and variable pollutant loadings is available with the CANNET model. Natural conditions, such as tidal harmonics, constant or variable freshwater or pollutant lateral inflow at specified locations, wind, initial concentration, and constant values for dispersion coefficients, vertical mass transfer coefficient, rate of exchange of pollutant at the tidal entrance, natural decay of concentration, and background concentration, may be specified for a given design. If necessary, the secondary flow induced by channel beds may be included in a three-dimensional configuration.

The results obtained from the evaluation of the example canal network indicate that the canal networks should be evaluated using realistic freshwater and pollutant loadings and variable winds for the specific site and the specific canal network configuration under consideration. Comparisons based on fixed winds and fixed pollutant loadings are useful only in evaluating relative effects in portions of the network.

The cost of running the model is moderate. Thus, it is practical to compare the results of several different design configurations in terms of the time required to flush the canal network, and to test the design for its response under a variety of natural conditions.

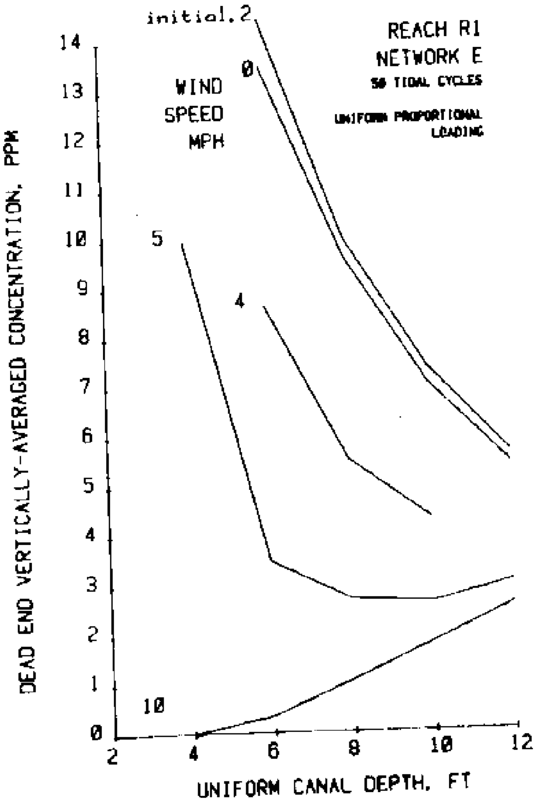


Figure 12.19a - Vertically Averaged Concentration at Dead-End of Reach R1, Network E, for Various Uniform Depths and Wind Speeds, Wind from the East.

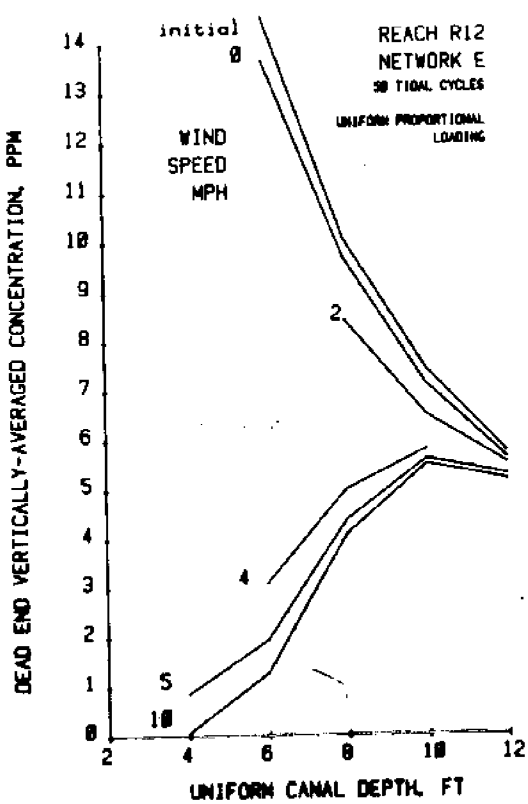


Figure 12.19b - Vertically Averaged Concentration at Dead-End of Reach R12, Network E, for Various Uniform Depths and Wind Speeds, Wind from the East.

CHAPTER 13
DESIGN ALTERNATIVES

It has been shown that some of the traditional methods for canal design have little potential for providing effective flushing. The limited energy available in small amplitude tides does not permit an inefficient hydrodynamic design to function properly. The energy that is available, however, on a day to day basis, supplemented by periodic inputs of from the winds, is sufficient, in many instances, to provide the required energy for flushing. This can be accomplished by:

- eliminating unnecessary energy losses
- utilizing the natural characteristics of open channels to optimize mixing where desired.

Unnecessary energy losses occur whenever there are sudden changes in channel geometry. There are some losses associated even with gradual expansions and contractions in a channel, but these are not nearly as severe as the losses caused at right-angle bends, deep holes, bridge piers, culverts, and docks. Energy losses also need not be restricted to the mechanical energy associated with hydrodynamics. For example, flushing is assisted by proper biological balances, such as the natural filtering action and nutrient uptake provided by vegetation and aquatic organisms, and the absence of vegetation and marine life represents significantly less available energy.

The natural characteristics of open channels that may be used to optimize the distribution of mixing energy include roughness elements, bends, variations in geometry with distance from the tidal entrance, shallows, surface area and sloping banks, turnovèr response to storms, tidal phase differences at two or more entrances, freshwater inflow at salinity dams, and others. A good canal network design will distribute any of these available characteristics to the best advantage of the system.

One of the most useful of these characteristics is the vertical mixing produced by helical flow. An example of a canal that takes advantage of this phenomena is shown in Figure 13.1. In this channel the direction of the helical flow will be reversed in each bend. This reversal will use some of the available energy, and for this reason may not be as efficient as the development shown in Figure 13.2. Here the helical flow persists in one direction all the way in to the marina on the flood tide.

A different application of curves or bends is shown in Figure 13.3. This system is simply an extension of the finger canal concept, but with the important features of multiple tidal entrances and relatively short canals.

In each of the three systems illustrated here, there has been much emphasis on including stabilizing vegetation, such as mangrove and spartina. The use of vegetated, sloping banks will provide many benefits, such as

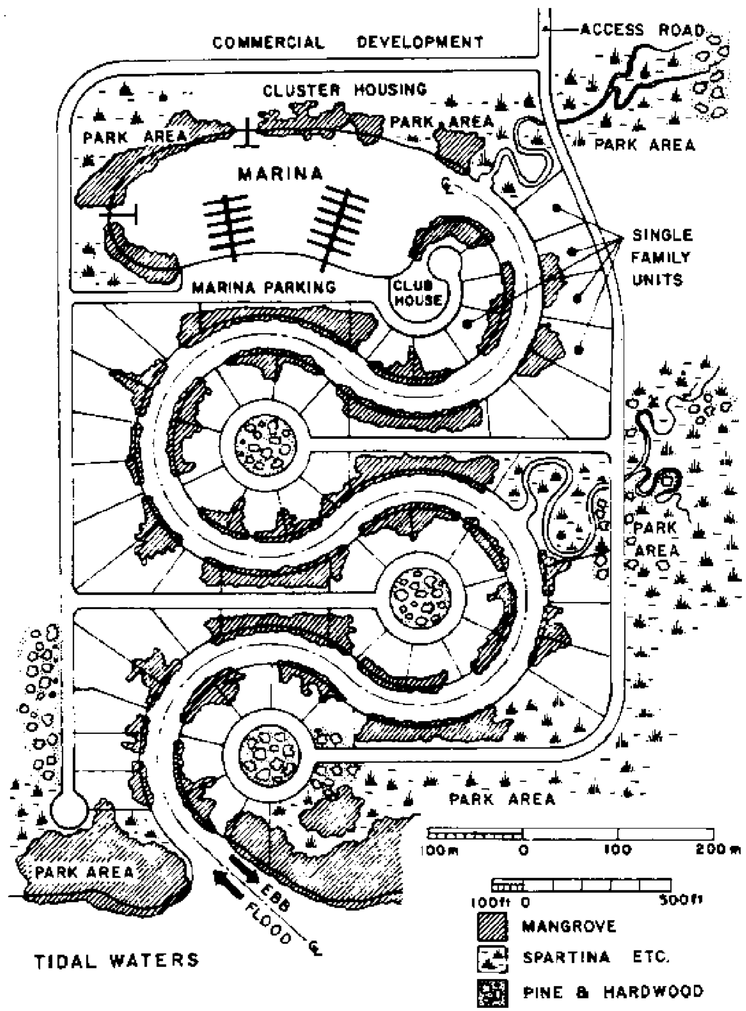


Figure 13.1 - Proposed Canal System With Reversing Bend.

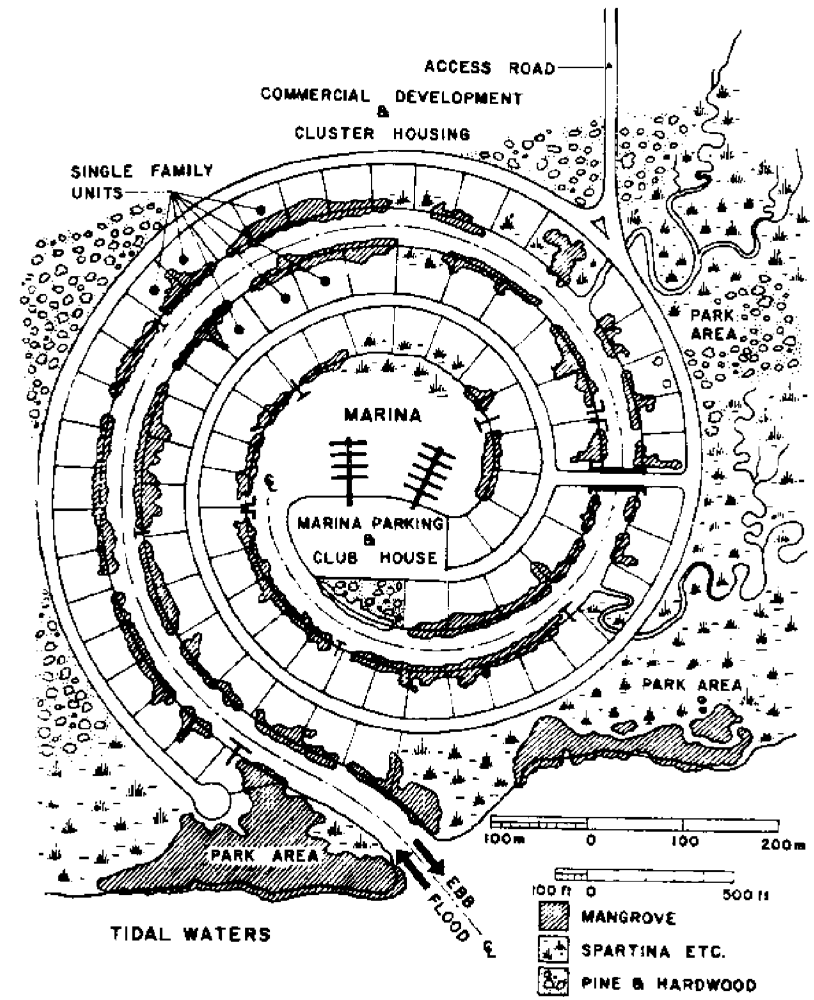


Figure 13.2 - Proposed Canal System With Spiral Bend.

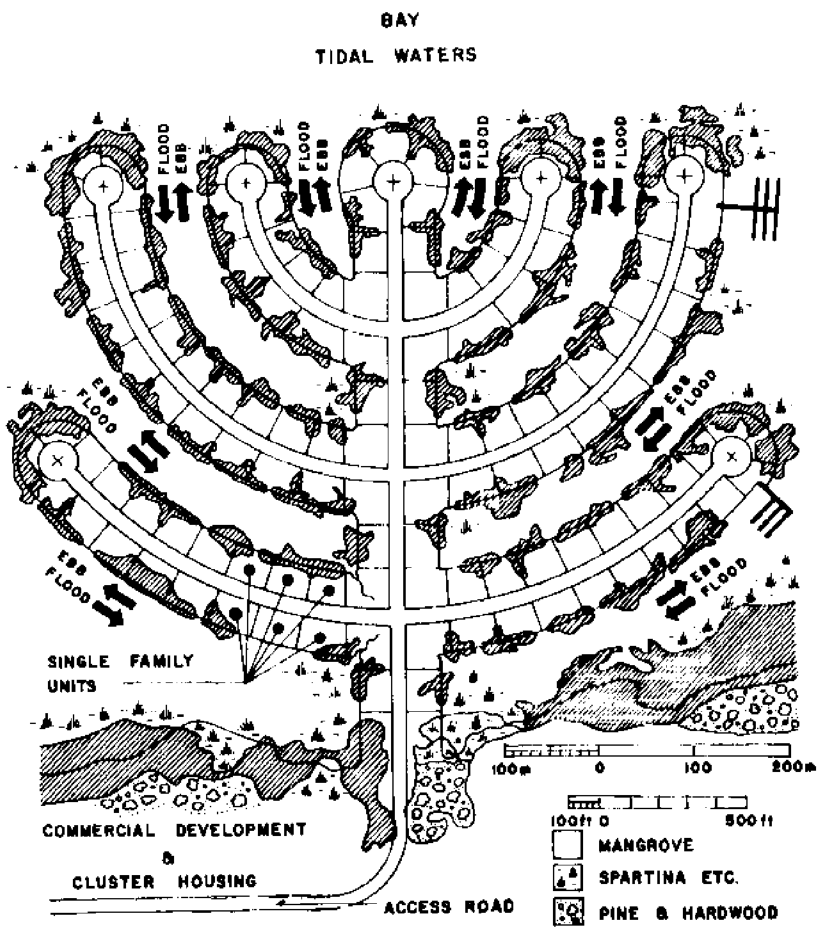


Figure 13.3 - Canal System With Bends and Multiple Tidal Entrances.

increased water surface area, effective bank stabilization, greater areas of intertidal shallows, and a more natural appearance to the area.

In the two systems which lead relatively far away from the tidal entrance a basin provides both an increase in the tidal prism and a marina. The basin will not have to be very large to ensure substantial tidal currents in the canal.

A final drawing (Figure 13.4) illustrates a more extensive application of the basic principles of sound hydrodynamic design. This is a canal designer's plan for a canal network currently being considered along the intracoastal waterway south of Jupiter, Florida. The plan includes:

- meandering channels
- large areas of intertidal shallows
- sloping, vegetated banks
- elimination of dead-ends

- increased tidal prism
- freshwater flow over salinity structures
- more uniform change in section through tidal entrances
- natural preserves set aside along the waterway.

The rational approach to canal design includes common sense planning, in-depth data collection, the correct application of physical, chemical, biological and ecological principles, and the use of judgement. The method cannot guarantee that a given design will function as planned, but it will provide the kind of guidance needed for environmentally compatible development.

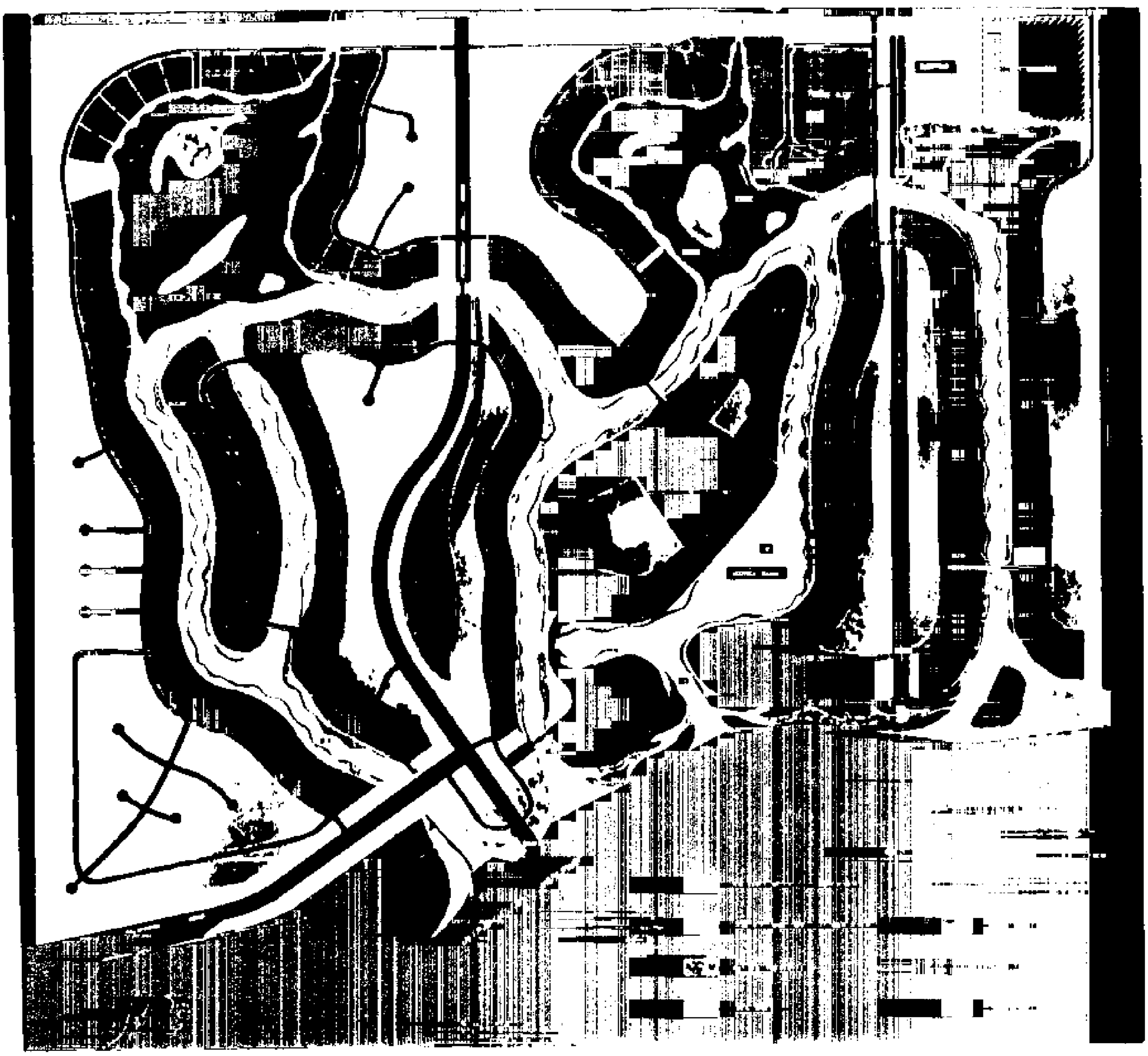


Figure 13.4 - Plan for a Canal System Designed With the Rational Method.

REFERENCES

Aris, R., 1956. "On the Dispersion of a Solute in a Fluid Flowing Through a Tube," Proceedings, Royal Society of London, Series A, Vol. 235, No. 1200, Apr. 1956, pp. 67-77.

Barada, W., and Partington, W. M., Jr., 1972. "Report on the Investigation of the Environmental Effects of Private Waterfront Canals," Report for State of Florida Board of Trustees of the Internal Improvement Trust Fund, Feb. 1, 1972.

Bureau of Coastal Zone Planning, 1977. "The Florida Coastal Mangagement Program. Workshop Draft," Dept. of Environmental Regulation, Tallahassee, FL, Oct. 1977.

Christensen, B. A., 1971. "Incipient Motion on Cohesionless Channel Banks," Ch. 4, Symposium to Honor Professor H. A. Einstein, Berkeley, CA, June 1971.

Christensen, B. A., 1973. "Design of Scour-Free Channels on Mild Slopes," Proceedings of International Symposium on River Mechanics, IAHR and Asian Institute of Technology, Bangkok, Thailand, January 1973, pp 749-760.

Christensen, B. A., 1976. "Mild Slope Stable Channels: A Stochastic Design Approach," ASCE Annual Convention, San Diego, CA, April 1976.

Christensen, B. A. and Snyder, R. M., 1978. "Establishment of Residential Waterfront Property by Construction of Canal Systems in Coastal Wetlands. Problems and Solutions," Paper 1301, Coastal Zone 78 Conference, San Francisco, CA, March 1978.

Clark, J. R., 1977. Coastal Ecosystem Management, John Wiley and Sons, New York, NY, 1977.

Cooper, C. K., and Pearce, B. R., 1977. "A Three-Dimensional Numerical Model to Calculate Currents in Coastal Water Utilizing a Depth Varying Vertical Eddy Viscosity," M.S. Thesis and Report 226, Parsons Laboratory, M.I.T., Cambridge, MA, Aug. 1977.

Corps of Engineers, 1977 "Regulatory Program of the Corps of Engineers," Part II, July 19, 1977, Federal Register, Washington, D.C., 1977.

Corps of Engineers, Florida DER, and Florida DNR 1977. "Joint Permit Application. Dredge and Fill Structures," Tallahassee, FL, 1977.

Eckenfelder, W. W., Jr., 1970. Water Quality Engineering for Practicing Engineers, Cahners Books International, Inc., Boston, MA, 1970.

Elder, J. W., 1959. "The Dispersion of Marked Fluid in Turbulent Shear Flow," Journal of Fluid Mechanics, Vol. 5, Part 4, Cambridge, England, May 1959, pp. 544-560.

Environmental Protection Agency, 1974. Effects of Exhaust from Two-Cycle Outboard Engines," NTIS NO. PB-223-567, Rensselaer Polytechnic Institute, 1974.

Environmental Protection Agency, 1975a. "Finger-Fill Canal Studies, Florida and North Carolina," Report EPA 904/9-76-017, Surveillance and Analysis Section, EPA, Athens, GA, May 1975.

Environmental Protection Agency, 1975b. "Navigable Waters. Discharge of Dredged or Fill Material," Part II, Sept. 5, 1975, Federal Register, Washington, D.C., 1975.

Fischer, H. B., 1967. "The Mechanics of Dispersion in Natural Streams," Journal of the Hydraulics Division, ASCE, Vol. 93, HY6, Nov. 1967, pp. 187-216.

Harleman, D. R. F., and Lee, C. H., 1969. "The Computation of Tides and Currents in Estuaries and Canals," Tech. Bul. No. 16, Committee on Tidal Hydraulics, Corps of Engineers, U. S. Army, Sept. 1969.

Lane, E. W., 1955. "Design of Stable Chanels," Transactions ASCE, Vol. 120, Paper Number 2776, 1955.

Lindall, W. N., Jr., Fable, W. A., Jr., and Collins, L. A., 1975. "Additional Studies of the Fishes, Macroinvertebrates, and Hydrological Conditions of Upland Canals in Tampa Bay, Florida," Fishery Bulletin, Vol. 73, No. 1, Jan. 1975, pp. 81-85.

Morris, F. W. IV, Walton, R., and Christensen, B. A., 1977. "Evaluation of a Hybrid Computer Model of Pollutant Flushing in Tidal Canals," Report to Florida Sea Grant, Project No. R/OE-4, Hydraulic Lab., Dept. of Civil Engr., Univ. of Fla., Gainesville, FL, Feb. 1977.

Morris, F. W. IV, Walton, R., and Christensen, B. A., 1978. "Hydrodynamic Factors Involved in Finger Canal and Borrow Lake Flushing in Florida's Coastal Zone," Report HY-7801, Final Report to Florida Sea Grant, Hydraulic Lab., Dept. of Civil Engr., Univ. of Fla., Gainesville, FL, March 1978.

Nordin, C. F., Jr., and Sabol, G. V., 1974. "Empirical Data on Longitudinal Dispersion in Rivers," Water-Resources Investigations 20-74, U. S. Geological Survey, Washington, D.C., August, 1974.

Piccolo, J., 1976. "A Guide for the Use of Tidal and Geodetic Datum Planes," Snyder Oceanography Services, Jupiter, FL, May 1976.

Polis, D. F., 1974. "The Envionmental Effects of Dredge Holes: Present State of Knowledge," Report to Water Resources Administration, Dept. of Natural Resources, State of Maryland, May 1974.

Public Health Service, 1967. "Manual of Septic Tank Practice," Pub. 526, U. S. Dept. HEW, in Cooperation with Joint Committee on Rural Sanitation, U. S. Government Printing Office, Washington, D.C., 1967.

Shields, A. "Anwendung der Ahnlichkeitsmechanik und der Turbulenzforschung auf die Geschiebebewegung", Mitt. der Preuss, Versuchsanstalt fur Wasserbau und Schiffbau, Berlin, 1936.

Snyder, R. M. 1976a. "57 Acres Project Development Report," prepared for Royal American Realty, Inc., Snyder Oceanography Services, Jupiter, FL, May 1976.

Snyder, R. M., 1976b. "Residential Canals and the Environment," Snyder Oceanography Services, Jupiter, FL, Oct. 1976.

State of Florida. Pollution of Waters, Chapter 17-3, 403 Florida Statues.

Taylor, Sir G. I., 1954. "The Dispersion of Matter in Turbulent Flow Through a Pipe," Proceedings of the Royal Society of London, Series A, Vol. 223, No. 1155, May 1954, pp. 446-468.

Walton, R., 1976. "Mathematical Modeling of Pollution Transport in Floridian Canals," IAHR Symposium on Unsteady Flow in Open Channels, University of Newcastle-Upon-Tyne, in association with BHRA, April 1976.

Ward, G. H. and Espey, W. H. "Estuarine Modeling: An Assessment," EPA, WQO Project 16070 DZV, PB-206 807, Tracor, Inc., Austin, TE. Feb. 1971.

The State University System of Florida Sea Grant College is supported by award of the Office of Sea Grant, National Oceanic and Atmospheric Administration, U.S. Department of Commerce, grant number NA80AA-D-00038, under provisions of the National Sea Grant College and Programs Act of 1966. This information is published by the Marine Advisory Program which functions as a component of the Florida Cooperative Extension Service, John T. Woeste, dean, in conducting Cooperative Extension work in Agriculture, Home Economics, and Marine Sciences, State of Florida, U.S. Department of Agriculture, U.S. Department of Commerce, and Boards of County Commissioners, cooperating. Printed and distributed in furtherance of the Acts of Congress of May 8 and June 14, 1914. The Florida Sea Grant College is an Equal Employment Opportunity-Affirmative Action employer authorized to provide research, educational information and other services only to individuals and institutions that function without regard to race, color, sex, or national origin.

Copies available from:

Marine Advisory Program
Florida Cooperative Extension Service
GO22 McCarty Hall
University of Florida
Gainesville, FL 32611

This public document was promulgated at a cost of $2,396.80 or $2.397 per copy to provide engineering information on residential and canal networks. Cost does not include postage and handling.