HYDRAULIC MEASUREMENTS, DATA ANALYSIS, AND RATIONAL DESIGN PROCEDURES FOR RESIDENTIAL TIDAL CANAL NETWORKS

BY

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NOTATION

a - tidal amplitude, (L)

a_o - constant of integration

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

A_s - surface area of canal, (L^2)

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

$\bar{A}(k)$ - function defined by Equation (7.31)

b - bottom width, (L)

B - top width, (L)

$\bar{B}(z)$ - width correction factor, (dimensionless)

c - turbulent time mean concentration (dimensionless)

c_I - concentration of lateral inflow, (dimensionless)

c_{RW} - background concentration in receiving waterbody, (dimensionless)

c' - fluctuation from turbulent time mean concentration, (dimensionless)

c_o - initial concentration, (dimensionless)

c_* - concentration at or near peak, (dimensionless)

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

d - depth, (L)

d_b - thickness of bottom layer, (L)

d_o - mean tidal depth, (L)

D - depth of reach, (L)

D_o - offset distance of bend from straight center-line, (L)

e_{tx} - longitudinal turbulent mass flux or diffusion coefficient, (L^2/T)

e_{ty} - lateral turbulent mass flux or diffusion coefficient, (L^2/T)

e_{tz} - vertical turbulent mass flux or diffusion coefficient, (L^2/T)

e - exponential constant - 2.718

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

- potential energy, (FL)

E_k - layer averaged vertical dispersion coefficient, (L^2/T)

$\tilde{E}_k$ - layer averaged, Richardson number dependent, vertical dispersion coefficient, (L^2/T)

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

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

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

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

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

E_γ - photon energy, (ML^2/T^2)

f - Coriolis parameter, (1/T)

- frequency, (1/T)

$F(\ )$ - exact solution of partial differential equation

$\tilde{F}(\ )$ - numerical approximation to partial differential equation

$F_1(\eta)$ - function defined by Equation (7.34)

$F_4(\eta)$ - function defined by Equation (7.35)

$\bar{F}_1(k)$ - layer averaged form of $F_1(\eta)$

$\bar{F}_4(k)$ - layer averaged form of $F_4(\eta)$

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

h - Planck's Constant, (FLT)

- 6.6256×10^{-34} Js

i - number of reach (Chapter 7)

- number of segment

j - number of reach (Chapter 7)

- number of lateral layer

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

- number of vertical layer

K - dimensionless dispersion coefficient

- decay coefficient, (1/T)

K_o - constant associated with an initial value (Chapter 9), (dimensionless)

K_R - reach uniform decay coefficient, (1/T)

K_w - wind drag coefficient, (dimensionless)

ℓ - length scale of turbulent eddy, (L)

- a characteristic length of the cross-section of a canal, (L)

L - length of reach, (L)

- length scale of convective period, (L)

- distance between injection point and sampling point

L_d - length of decay of secondary current, (L)

L_w - length of saltwater wedge, (L)

M - mass of pollutant released, (M)

N - number of tidal cycles

N_a - Avogadro's number (mol^{-1})

- 6.02252×10^{23} mol^{-1}

Nu - number of upstream reaches Equation (7.1.2)

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

$\bar{N}_z$ - constant defined by Equation (7.26)

p - variable used in Section 7.1.3

- permissible deviation from background velocity Equation (7.40)

P - power available from tidal prism, (FL/T),

P_a - atmospheric pressure, (M/LT^2)

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

Q - discharge, (L^3/T)

Q_u - discharge at upstream section of reach, (L^3/T)

$\bar{Q}$ - mean discharge, (L^3/T)

r - radius of bend, (L)

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

R - hydraulic radius, (L)

- width of volume in cell after convective step, (dimensionless)

Ri - Richardson number, (dimensionless)

R_m - dimensionless width of distribution in cell

s - inverse side slope, (dimensionless)

s_L - inverse side slope of left bank, (dimensionless)

s_R - inverse side slope of right bank, (dimensionless)

t - time, (t)

- temperature (°C)

t_p - elapsed time to measurement of peak concentration, (T)

t' - time since low tide, (T)

t_* - time corresponding to c_*, (T)

T - tidal period, (T)

T_F - flushing time, (T)

T_p - time to peak, (T)

T_R - mean residence time, (T)

$\bar{U}$ - mean, steady, uniform velocity of flow, (L/T)

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

u_{D_x} - dispersion velocity in x-direction, (L/T)

u_{D_y} - dispersion velocity in y-direction, (L/T)

u_{D_z} - dispersion velocity in z-direction, (L/T)

u_F - velocity of front of saltwater wedge, (L/T)

u_{sm} - mean velocity in the salt wedge, (L/T)

u^* - bed shear velocity, (L/T)

u' - turbulent fluctuation from time mean velocity in x-direction, (L/T)

u_Δ - densimetric velocity, (dimensionless)

u_4 - constant defined by Equation (7.43)

v - lateral velocity component (L/T)

v_{ck} - layer averaged secondary current, L/T)

V - volume of tidal prism upstream of section, (L^3)

V_w - transfer volume due to wind, (L^3)

W - width of reach, (L)

w - vertical velocity component, (L/T)

w_s - wind speed, (L/T)

w' - turbulent fluctuation from upstream section of reach, (L)

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

x' - distance from tidal entrance, (L)

X - distance, (L)

XD - longitudinal dispersion distance, (L)

y - lateral coordinate direction

- depth, (L)

z - vertical coordinate direction

Greek Letters

α - included angle between radii to ends of a curved reach, (rad)

γ - unit weight of water, (F/L^3)

∂ - partial derivative operator

Δ - change

Δt - time increment, (T)

Δx - longitudinal spatial increment, (L)

Δy - lateral spatial increment, (L)

Δz - vertical spatial increment, (L)

$\Delta\rho, \Delta\rho_o$ - incremental density, (M/LT^2)

η - elevation of water surface from the mean depth, (L)

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

κ - von Karman's constant = 0.4 (dimensionless)

π - universal constant = 3.141593

ρ, ρ_o - density, (M/L^3)

σ^2 - variance

σ_t - density function used in oceanography

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

τ_{xs} - surface longitudinal wind shear stress, (M/LT^2)

τ_{xx} - shear stress in x-direction with respect to x-direction, (M/LT^2)

τ_{xy} - shear stress in y-direction with respect to x-direction, (M/LT^2)

τ_{xz} - shear stress in z-direction with respect to x-direction, (M/LT^2)

τ_o - bed shear stress, (F/L^2)

$\phi(Ri)$ - function of Richardson number, (dimensionless)

ω - tidal frequency, (1/T)

Subscripts

av - average over two time layers

b - bottom layer

f - variables in freshwater layer above saltwater wedge

ℓ - longitudinal direction

LT - low tide

m - spatial mean value

max - maximum value

p - previous time level (Section 7.2.3)

RW - receiving waterbody

s - variables in saltwater wedge

- surface

- speed

t - tidal variables

TE - tidal entrance

w - wind variables

0 - node at upstream section of reach

1 - node adjacent to upstream node of reach

- base of layer

-1 - base of top layer

2 - node, two away from upstream node of reach

- top surface of layer

Superscripts

n - time level

$\bar{}$ - turbulent time mean value

$\tilde{}$ - value of variable at intermediate step

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A rational approach to the design of residential canal networks through the formulation of design objectives, guidelines, and quantitative criteria is developed. The legislative constraints currently in effect are summarized, and recent environmental guidelines relevant to the design of residential tidal canal networks are reviewed, particularly as they apply to Southeast and Gulf states. An overview of the important climatological, hydrodynamic, and water quality features of canal systems is given as a basis for understanding the hydrodynamics of tidal canals and tidal canal networks. A general approach to canal design through the formulation of design objectives, guidelines, quantitative criteria, and engineering constraints is outlined. The evaluation of canal site characteristics is described, together with appropriate measurement requirements, techniques, and instrumentation and support equipment.

Field measurements conducted during the development of the canal design project are shown to have had a fundamental influence on the

development of a three-dimensional mass-transport mathematical model for canal networks. The characteristics of the model, a separate project developed simultaneously with the canal design project, are described. The unique features of the model include its capability of modeling wind-induced circulation, density currents associated with a saline wedge, and helical flows induced by bends in the canals. Field measurements conducted from 1974 through 1977 are summarized, and their implications are discussed. The verification of the model with field data is demonstrated to be consistent with data from two different canal systems in Florida.

The variability of an equilibrium concentration profile with canal geometry, a time constant associated with transport of mass away from the tidal entrance, lateral inflow of mass along the canals, roughness, and the longitudinal dispersion coefficient are shown by numerical simulation. The response of simple canal network design elements, in terms of rate of flushing, to changes in channel depth, speed and direction of wind, and tidal prism are shown. The flushing of canals and canal networks is demonstrated to be primarily dependent on the wind. The design alternatives for the improvement of flushing in a traditional residential canal network are illustrated by means of an example.

CHAPTER 1

INTRODUCTION

Florida's rapid growth over the past three decades has been accompanied by a large demand for residential dwellings in the coastal zone. In particular, the demand for waterfront property has led to the dredging of many residential canal networks along both the Atlantic and Gulf Coasts, and these canals have often been built without consideration of the natural conditions at the site. Large canal projects constructed in ecologically sensitive areas have destroyed aquatic nurserys, removed natural barriers to storm tides, disrupted surface water flows, and polluted bays and coastlines. As a result, canals have received a sinister reputation which, while deserved in many cases, should not be universally applied. As shown herein, canals networks can be designed to maintain their water quality without adverse effect on the environment, using only natural forces.

1.1 Background

The story of residential canal development in Florida begins early in this century, when the state was still considered a frontier and men were preoccupied with converting the land to a more habitable and commercially useful form. Residential canals were not being constructed in those years, but patterns of land use were developing which would intensify through several stages of growth and culminate in the grandiose waterfront land development schemes of the 1950's and 1960's.

Florida is a subtropical peninsula overlying a deep, layered, limestone aquifer. Its coastal lowlands extend far inland, and its southern part is dominated by vast wetlands which extend westwards to a marshy coastline. This marsh and swampland has been viewed by successive generations of inhabitants as an impediment to progress, sometimes even as a wasteland, which had to be conquered and transformed to "useful" property. This perceived need developed into a dream which, supported by money and the policy of the state and federal governments, spawned the Everglades drainage canal projects (begun in 1880), federal flood control canal projects (1907 through 1970), and the residential canal projects following the Second World War. Thus, during the first fifty to seventy-five years after statehood had been acquired in 1845, trends in land use were being established which would not be changed until the "environmental decade" -- the 1970's. During the early years of statehood the development of remote areas and wetlands was encouraged by federal and state governments through direct sales and grants of federal and state lands to homesteaders.

In 1869 the completion of the railway to Miami provided a transportation link with the north which would bring thousands of settlers and eventually hundreds of thousands of tourists into the state, and would carry agricultural products out to the markets in the rest of the country. In 1911 the "first swamp salesman" [Carter, 1974, p. 69], Richard J. Bolles, introduced the contract method of selling, on an installment basis, land that had been reclaimed by the state and subdivided, but was unimproved. This enterprise, which resulted in many law suits and investigations, became a national scandal. The state was then faced with a dilemma; if it desired to continue the drainage of

wetlands, it could do so only by promoting the sale of state lands. But these sales easily attracted gullible buyers, and seemed by their very nature to encourage fraud. The decision was made, drainage continued, and a policy was established which led inexorably to substantial alternation of Florida's natural features.

In 1913 a promoter from Indiana, Carl Fisher, who had bought a home in Miami, began what was to become the first large scale dredging and filling project in Florida. This was located on Miami Beach, then a long barrier beach across Biscayne Bay from Miami. To complete this project a thousand acres of mangrove were filled with six million cubic yards of bay bottom [Redford, P., 1970, in Carter, 1974, p. 75]. While this was not a canal development, the dredge and fill technique was demonstrated by this venture to be both feasible and profitable.

While the state was carrying out its plan for drainage of the Everglades, and Fisher was constructing waterfront lots on Biscayne Bay, another large-scale project was taking shape in south Florida. Barron G. Collier, a New Yorker who had a winter home near Fort Myers on the west coast, was buying large tracts of land in and around the Big Cypress watershed southwest of Lake Okeechobee. In return for a promise to complete the Tamiami Trail, a road across the Everglades between Tampa and Miami, he was encouraged by the state to pursue his plans for development of his 900,000 acre holdings. The highway was completed in 1926 by the State Road Department after eleven years of tortuous work in the swamps, but little development was accomplished in this portion of the state until the 1960's.

In 1962 draglines, bulldozers, and tree-crushers began extensive alteration of a site near the coast, in what had by then become Collier

County. By 1974 an irregular area had been leveled for a distance of twenty-five miles north and south across the Big Cyrpess swamp, and major canals, discharging incredible volumes of freshwater from the development area, had been opened to Naples Bay, Rookery Bay, and Fahka Union Bay. By 1974 a grid of 171 miles of undeveloped canals and 807 miles of undeveloped roads had been constructed on this property. These smaller canals were connected to the Fahka Union Canal ten miles upstream from its mouth and to the Golden Gate Canal three miles from Naples Bay [Carter, 1974, pp. 236-240].

A study of the area by the Environmental Protection Agency (EPA) [1973] showed that dredging had drastically increased runoff to the canals, thereby decreasing the area of potential inundation by surface waters during the wet season with subsequent undesirable ecological side effects [EPA, 1973, p. II-3]. The canals had lowered the groundwater table by two to four feet, significantly increasing saltwater intrusion; intercepted surface flow and drastically decreased the time for surface water to reach the receiving water, which affects ecosystems dependent on a steadier supply of freshwater and diverts minerals and nutrients directly to the estuarine waters; reduced primary productivity in cypress forests and wet prarie ecosystems; increased the drying rate on the forest floor, leading directly to increased spreading of wildfires; and caused subsidence of organic soils [EPA, 1973, Chapters I through VII]. While these effects are extreme, due to the immense size of the development and its ecologically sensitive location, they are nevertheless typical and illustrate many of the adverse effects often associated with poorly sited and improperly designed canal developments.

During the first half of the twentieth century, then, development in

Florida was encouraged by the state and was largely unregulated. After the Second World War a huge retiree market was discovered and feverishly exploited. The out-of-state market, particularly for waterfront property, continued to grow into the 1970's attracting persons approaching retirement, younger people making long-term investments toward retirement, and speculators [Carter, 1974, p. 29]. The sudden awakening of citizen consciousness to the environmental stress brought on by this exploitive style of development, and the subsequent introduction of protective legislation and enactment of new regulations stemming from the National Environmental Policy Act (NEPA, 1969), appears to have finally caused large developers to attempt to design pleasant and environmentally acceptable new communities.

The large-scale developments, some of which were conceived before the "environmental decade", are listed in Table 1.1 [Florida Trend Magazine, June, 1974, in Carter, 1974, pp. 32-33]. This shows the extensive acreage involved and the current and ultimate population figures for which the developers were planning. Not all of these developments are sited directly in the coastal zone, but many can be identified as residential canal developments and all will have a major effect on water resources.

Due to past abuses, caused in some instances by lack of knowledge of the consequences of crude construction practices, and in others by greed, present-day development regulations affect everyone, from owners of small properties to large land development corporations, who desires to alter a coastal area for any reason. Recently the regulatory process in Florida has been somewhat simplified to permit relatively insignificant dredge and fill for maintenance, and the construction of small

improvements, with a minimum of delay. The overall approach taken by the regulatory agencies in the 1970's, however, has been one of caution and deliberation.

In retrospect, it is not difficult to understand why such an attitude has evolved, nor can one find much fault with the intentions of the citizens and the government in this regard. While in the past some development in the coastal zone has been carried out with good judgement and accommodation of all the known environmental factors, examples of such development are few. The more spectacular and environmentally inconsiderate examples remain as major liabilities with regional effects that will continue to cost the citizens, the state, and the federal government much in terms of corrective measures and maintenance (for example, see Carter, [1974, pp. 236-240]).

1.2 Problems Associated With Canal Development

There are many ways in which canal systems can be classified. For a preliminary discussion of Floridian residential canals three principal types of waterfront canals may be distinguished, following Barada and Partington [1972] and Lindall and Trent [1975]:

1. *bay-fill* or *finger-fill* canals, which are constructed below mean low tide by dredging and filling shallow bay bottoms (Figure 1.1).
2. *intertidal* developments, which are 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, estauries, lakes, or other wetlands (Figure 1.2).

3. *inland* or *upland* canals, which are 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 1.3).

Residential canal systems are usually constructed by dredging in a manner which maximizes the density of housing lots and is most convenient for the developer, 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 have 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 Key Largo limestone substrates and higher elevation, upland Miami oolite 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. Dredge and fill is now carefully regulated in Florida, and spoil banks must be located where they cannot leach into tidal waters. But if the dredging process itself is not carefully controlled, bottoms may be cut through sediments into underlying bedrock, creating zones for transfer of denser saltwater and possible pollutants into the aquifer. Examples are cited by Griffin [1977, in Morris et al, 1977b, Appendix A, pp. 8, 10, 15]. In addition "wavy" longitudinal bottom profiles will

result if the dredge operator follows the tidally-fluctuating water surface as a reference level.

Two physical features which have been singled-out for particular attention by previous investigators are the depth of canal and the possible presence of a sill. A sill is often created when the canal is first dredged before connection to the "receiving" waterbody. In its most general form, however, a sill is a relatively shallow section at any location in the canal which impedes the circulation in the bottom waters inside the canal.

In general, it has been observed that "deep" canals are not adequately flushed by tidal action and that the "lower layer acts as a trap for sediments and organic detritus" [Polis, 1974, p. 21]. 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. This stratification was reported to be "apparently due almost entirely to depth, regardless of proximity to open water or canal configuration" [O'Hara, J., 1971 in Barada and Partington, 1972, p. 10].

Canals that are too shallow can also have poorer flushing characteristics, and limited navigability as well. Writing about residential canal systems in the Florida Keys, Chesher observed:

> Canals should be deep enough that boat traffic will not disturb the bottom and shallow enough for good biological productivity. This depth varies from one area to the next and depends on the substrate and the flushing characteristics of the canal. Five feet is probably too shallow for most areas. Canals

> of this depth have poor flushing characteristics over long distances... Boats can disturb the bottom in depths of less than five feet, thus increasing turbidity and damaging bottom communities. Chesher [1974, p. 2].

In searching for a simple method by which "good" and "bad" canals can be separated, governmental agencies have found that in general the vertical dissolved oxygen profile (or surface and bottom values) can be related statistically to the mean depth. Figure 1.4, for canals in Florida, and similar relationships for North Carolina canals [EPA, 1975b, p. 11; Walton, G.F., 1976, p. 142] shows such a trend, which seems to indicate that this is not a local condition. The 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, 1975b, p. 5]. The one-dimensional dispersion model used in making this determination, which is based on very restrictive assumptions, and the cases which were simulated, were not realistic enough to encourage wide accepance of this oversimplified criterion.

A sill in a canal acts as a trap for the bottom, denser water and fluidized sediments, and suppresses vertical mixing. Since vertical mixing is the principal means by which reaeration of the bottom waters is effected, the accretion of "flocculent sediments and organic detrital matter" [Polis, 1974, p. 38] 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. Effective wind mixing can lower this interface 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 which intrudes into the aquifer to an underground distance which depends on the freshwater head above the piezometric head line. As this head decreases, the saltwater interface moves upward and inward farther into the aquifer. This relationship is known as the Ghyben-Herzberg principle, from independent research on saltwater encroachment made by Badon-Ghyben [1888] in Holland and Herzberg [1901] in Germany. In their investigations into the equilibrium relationship between the shape and position of the freshwater/saltwater interface, a simple expression for the ratio of water table elevation above mean sea level to the interface depth below sea level was derived under simplifed but realistic conditions. Considering the difference in the density between fresh and saltwater, it was shown that the depth of the interface below mean sea level is about forty times the height of the freshwater table above it. The effect of a canal is to bring tidal waters farther inland, and sometimes also to 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 a 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 simply a 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 instances in 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. It is because circulation is the basic mechanism for maintenance of water quality that this canal design project has concentrated on a comprehensive description of the hydrodynamics in tidal canal networks.

The circulation or movement of water in a tidal canal network is governed principally by the geometry of the channels and the tide, the wind, and density gradients. In Florida the tidal effect is small compared to other coastal areas, the mean amplitude ranging from less than about 3 ft on the west coast to 2.5 ft or less in the Keys to about 2 to 3 ft on the lower east coast; since the tide is mixed, alternate tidal cycles are even smaller in amplitude. The tidal energy flux into a typical 80 ft wide Florida canal is about 4 hp per mile, corresponding to a 2 ft tidal range. This means that the energy available from the tides for mixing in Floridian canals is relatively small. However, if more than one tidal entrance can be provided, and if these entrances can be separated far enough to provide a tidal elevation differential, then flushing can be substantially improved.

Wind provides a surface movement which is often accompanied by a return flow in the lower layer, and sometimes a three-layered flow, which effectively increases vertical mixing. When winds are relatively steady and directed along the channel, the geometry of the channel induces secondary, helical currents which mix surface water downward and bring bottom water to the surface, also increasing vertical mixing. Density gradients, on the other hand, inhibit vertical mixing across the

density interfaces. There are, thus, several phenomena which combine to give a variety of circulation patterns, which once understood, can be used to advantage in designing a canal network which will optimize flushing throughout.

The favorable effect of wind on mixing in Floridian canals has been recognized in most studies of the causes of water quality variations. Alignment of the channels with prevailing land breezes can keep surface debris from collecting in finger canals and can induce a vertical circulation inward along the bottom, but canals are not usually aligned purposely with the wind. EPA recommended that "orientation of canals should take into account prevailing wind direction so that flushing/mixing would be enhanced and wind drift of floating debris minimized" [EPA, 1975b, p. 6].

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 they might be 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. Griffin [in Morris, et al, 1977b, Appendix A, p. 5] suggests that this could be the cause of a recurring fish kill in a Key Largo canal.

Another aspect of canal design which has been mentioned in the literature is bank and bottom stability. The effect of water velocity on suspension and deposition of sediments has been quantified for various channel cross-sectional, shapes, and is a factor in the analysis of inlet stability. Velocities which are too low are accompanied by 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. A given channel geometry is characterized by a stable cross-section which neither accretes nor erodes, and which adjusts itself on a long-term basis to changes in the quantity of water flowing. Thus, another element which must be considered in the canal design process is the stability of the channel.

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 lead to algal blooms, oxygen depletion, fish kills, and subsequently worsening conditions (a form of positive feedback).

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 canal 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 into 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 from storms. These pollutants are either flushed out, or accumulate on the surface or in bottom sediments, depending on the flushing ability of the canals. The quantity of urban runoff water can be reliably predicted from analyses

of soil characteristics and the percent of developed surface area, for various size storms in a given drainage basin, and some data are available for typical nutrient and bacterial contents of urban runoff. State legislation has established limits for many chemical constitutents for various classes of water.

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. This problem was particularly acute in the Florida Keys in the early 1970's; a land use planning study had determined that none of the soil in Monroe County was suitable for this mode of sewage treatment, while approximately 90 percent of the residences in the Keys were utilizing it [Smith, Milo, and Associates, 1970, in Barada and Partington 1972, p. 29]. 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. However, enforcement is difficult due to the many point sources that do exist, or can exist in a

new development. The EPA recommended that "no sewage plant effluent or other point-source discharges should be discharged directly into finger-fill canal waters. Discharges into surface waters should be sufficiently distant from the canals to ensure that the effluent is not carried into the canal systems by tidal currents" [EPA, 1977b, p. 5].

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. 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.

However, in a study of fifty canal systems in the Florida Keys, of which forty-four were man-made canals, Chesher arrived at the following conclusion:

> Disadvantages which have been alleged but which are unsubstantiated in the Florida Keys by this or other studies include such things as excessive nutrients from fertilizer runoff, excessive transmission of heavy metals and pesticides from residential areas into ambient water, widespread disruption of marine communities by increased turbidity from dredging activities, hazardous levels of septic tank seepage into canals, accumulation of organic muck on the bottom of canals, low dissolved oxygen levels from septic tank pollution and stagnation, and others. These allegations were investigated during this study but no evidence for their support could be found. Pesticide accumulation was found in some canals but was also present in natural man-made canals and was not obviously correlated with population density.
> [Chesher, 1974, p. 12].

Apparently, there has been some disagreement on the seriousness of this problem.

The major problems which have been identified with Florida's residential canal developments have been listed here along with some

conflicting data and opinions. In many instances these problems can be eliminated or reduced to meet state requirements with proper canal siting and channel design. The design, however, must take into account the existing site characteristics, and must be thorough in considering all possible effects on the water quality in the canal system as well as the ability of the canal system to maintain itself. In addition, the more extensive question regarding the canal system's effects on the water quality in the receiving waterbody must also now be considered, as required for any permit.

1.3 Present State of Canal Design

Snyder [1976b] has summarized the adverse effects of canal design criteria which are established for the convenience of the developer and without regard for the environment (Table 1.2). If design criteria comprise only navigable depths to the shoreline, maximization of front footage, increased lot elevation, minimum commitment of property to water (i.e. canal) area, rapid drainage of stormwater, and simplified surveying and construction methods, the results will almost certainly be destruction of habitat and degradation of water quality both in the canal system and its environs. It will subsequently be shown how reconsideration of canal design criteria can eliminate all of the objectionable features of residential canal systems, at properly selected sites.

Present canal design is often planned to primarily satisfy the objectives of the developer. His interests are chiefly economic, although he must also contend with new regulations requiring that water quality standards be met and there be no significant adverse effect on the environment. But what tools are available which will enable him to

predict the operational characteristics of his design? The regulatory agencies can evaluate a canal plan on a subjective basis from past experience, by looking at the gross features of the plan and noting those which obviously will not permit adequate circulation, or will destroy habitat, or which may introduce high levels of pollutants into the receiving waters. Crude calculations of flushing based on tidal prism methods and an assumption of homogeneity in the water column can be made. At best, up until now, existing one-dimensional models might be used to evaluate the circulation and concentration of conservative substances in simple canal geometries under no wind conditions. Such evaluations, based on inadequate models, can give grossly distorted predictions.

1.4 Objectives of the Canal Design Research Project

In 1975 the Office of Sea Grant, National Oceanographic & Atmospheric Administration and the Board of Regents of the State of Florida University System awarded a three-year contract to the University of Florida through the State University Sea Grant Program, supplemented by a three-year grant by the Board of Commissioners of Palm Beach County, to study the hydrodynamics and transport properties of residential finger canal networks in the coastal zone and to develop an objective canal design procedure and a canal design manual.

One of the principal objectives of the canal design research project was to describe the various characteristics of canals which determine their suitability or unsuitability in a variety of locations. A second overall objective was to develop a means whereby the operation of a canal system, either existing or planned, can be evaluated or predicted. With a predictive capability, and an optimizing procedure which will

permit a designer to improve his plan in a systematic manner, the design and evaluation of tidal canal systems can evolve from an essentially random, subjective process to an objective engineering process.

The research project was organized into five principal topics or sub-areas for investigation:

1. Evaluation of those characteristics of Floridian canals which must be considered in canal designs, and integration of these characteristics into the design process.
2. Field measurements and data analysis to support the modeling efforts, including an evaluation of the detail and extent of data collection required for design purposes.
3. Physical (hydraulic) modeling for determination of the basic hydraulic and pollutant transport characteristics of canal channels.
4. Numerical (computer) modeling for simulation of water circulation and dispersion of pollutants in canal systems.
5. Decision modeling for evaluating canal performance and finding optimal canal network designs.

These topics, which are discussed individually in more detail in the following chapters, are closely interrelated. The physical, climatic, geological and biological characteristics of a canal site and the quality of the water (both existing and predicted) constitute some of the primary limiting factors in the canal design, and affect every facet of planning for a development. Initially, each of these factors must be properly described and interrelated, and the interrelationships between the canals and these factors must be considered at every step of

the canal evaluation process.

The design of a new canal system, or the redesign of an existing canal system, requires certain specific measurements at the site. For either application, the types of field data required may be categorized as follows:

1. data for an analysis of the performance of an existing system
2. data for input into the hand calculations and numerical modeling which are part of the design process, consisting of:
   a) data defining the geometry of the existing system,
   b) data defining the ranges and the interrelationships of the forcing functions (tide, wind, and salinity and temperature gradients),
   c) data for calibrating the numerical model (the forcing-function data plus measurements of dispersion).

One of the most important objectives of this study was to review the types of instrumentation available and to recommend those types which would be most suitable for the identified data requirements.

The field measurement portion of a given design project affects, to a significant degree, the quality of every other portion of the study, as well as the final recommendations. Very little substantive work can be accomplished without correct and complete data on all significant variables, and extrapolation of results beyond the bounds of the data base can be quite risky. Therefore, the plans for data acquisition must be made carefully and with all due consideration of the existing conditions at the site and the needs of the designer.

Physical modeling is essentially a research tool used for determining basic flow and dispersion characteristics for theoretical, simplified situations. Also, physical models are very useful for complementing research conducted with numerical models, since one type of model can simulate effectively at different levels of spatial detail and time than the other. While hydraulic modeling can be a very useful design tool, it is an expensive and time-consuming approach for the one-time evaluation of a particular project and cannot be considered as a tool which could be made available on a practical basis to canal designers [Morris, Walton, and Christensen, 1977a]. Instead, numerical models are more flexible and more easily used by engineers, designers, and planners.

Numerical modeling is the technique used in this project for simulating the operation of a canal design. Once a model has been calibrated and verified, it provides the means by which changes in a given design can be assessed.

Decision modeling provides the designer with an ability to improve his design in steps which will optimize one or more selected parameters. For example, if a given design does not provide an adequate degree of flushing, this decision capability will guide the designer to a modification in the design which will improve the flushing without recourse to random trial and error methods. The objective of this portion of the canal research project, therefore, was to develop the necessary facilities for ensuring that designers, planners, and regulatory engineers can use the numerical models effectively in developing optimal canal designs. The effectiveness of this portion of the project obviously depended upon the quality of data and the numerical model.

1.5 Field Observations

The objectives of the field work associated with this poject were:

> to determine the significant features of tidal canal hydrodynamics and the transport of substances by convection and diffusion, for purposes of developing a numerical mass-transport model.
>
> to develop effective measurement techniques and determine the types of instrumentation and support equipment required for evaluating development sites and existing canal networks.
>
> to obtain the data necessary for verification of the numerical model for particular sites

Depending on the information and data required at a particular phase of the canal design, field operations may be conveniently divided into "preliminary site investigations" and field surveys". The former refers to field work which is required for obtaining qualitative planning information about the site. This type of information includes any conditions which will limit field measurements, unusual conditions which may require additional measurements or special equipment, and locations of benchmarks for surveying. Field surveys, on the other hand, are designed to obtain quantitative information such as the actual magnitudes of physical and environmental data for design calculations and numerical models. Both of these general types of field observations have been conducted by the author in support of the canal design project.

In addition to the principal investigator and the numerical modeler, it is advisable to include in the site investigation team several other well qualified scientific persons to provide opinions on characteristics not familiar to the principal investigator. A biologist or ecologist should be available to make observations relating to water quality, aquatic life, and vegetation; a geologist to look at soils, sediments,

and the geologic structure of the site; an oceanographer or coastal engineer who can relate shoreline topography to the features of the water circulation; and a representative of the State Department of Natural Resources, to provide comments on the suitability of the site for development. The developer and the canal designer should not assume that they can by themselves learn all they need to know about a site, but instead can learn much from people with qualifications in other scientific disciplines.

There are basically two kinds of information and data that can be obtained for a site. Usually a search will reveal earlier reports on the same area, or perhaps at the same site, which may provide useful historical perspective. Particularly useful are data on previous landforms, including land elevations and waterbody depths, and water quality, which may reveal trends toward improvement or degradation at the site. Aerial photographs are also a good source for historical information. The investigator should obtain the chronology of land ownership, which may lead to additional information about past conditions at the site.

There is a need for both long-term field surveys and short-term, intensive field surveys. Most natural variables, such as tide, wind and rain have long as well as short periodicities which may be significant. Some variables, such as water velocity, salinity, and temperature structure in a canal, need to be interrelated on a short period basis during a tidal cycle, and these variables require short, intensive surveys.

The principal investigator and the field survey team need to have a definite, well-organized plan before any survey is conducted. This will ensure that no essential measurements are missed, at least due to lack of planning.

1.6 Numerical Modeling

Today there are many numerical models available for simulating hydrodynamics, transport phenomenon and water quality for almost every conceivable type of waterbody. A number of these models have been used so often that the results obtained are, unfortunately, accepted without question. This has led, in some instances, to the application of models to problems for which they were completely unsuitable, since each model is designed under a specific set of assumptions and for a specific range of variables which may be entirely different from those required for the problem at hand. In addition, there has also been some tendency to develop both oversimplified models, and over-elaborate models in which complex techniques are used to approximate terms which have only a relatively small influence in a particular situation, and which therefore can usually be neglected considering the quality of the input data or the accuracy of the other inherent approximations and assumptions.

A distinction is usually made between hydrodraulic or hydrodynamic models and water quality models, because they are usually developed independently by different persons working in substantially different disciplines. The user of the models will also frequently have developed his expertise and experience in a discipline related to one or the other of these areas, but not both. Thus, the limits within which each model has been designed, that are established by the basic assumptions in the mathematical development, the approximations inherent in the numerical method, and the accuracy of the field data used in calibrating the model, may not be fully appreciated by the user.

For example, a frequent over-simplification is to assume that the

hydrodynamics can be reproduced by a one-dimensional model. This would be a fatal mistake for the analysis of canal networks, in which external influences such as the wind and density gradients produce multi-layered flows which are predominant over tidal influences. Similarly, some hydrodynamic models are too elaborate for this application. For example, models based on the full dynamic equations are not required when the surface slope of the canal discharge is very small, as it is in the low energy tidal canals characteristic of Florida's coastal zone.

Thus, one of the principal goals of the numerical modeling portion of the canal design research project was to develop a predictive three-dimensional model that would incorporate the physically important factors in canal hydrodynamics and mass-transport. Furthermore, the model had to be relatively inexpensive to run, so that it would be feasible to iterate a canal design through trial configurations toward an optimal solution. The input of boundary conditions, such as the tide and fresh water or pollutant inflows, had to be simple, but flexible enough to accommodate a wide variety of possible applications. Likewise, the specification of the other forcing functions had to be flexible to permit either time-carrying field conditions or fixed design conditions to be applied.

The calibration of a numerical model for a particular site can often be a very difficult task, relying on a great amount of field data. A second goal of the numerical modeling portion was therefore to structure the model with a minimum of calibration coefficients. A model that fulfills the goals of the project was developed by R. Walton [1978].

1.7 Organization of The Chapters

The principal features of typical tidal canal systems in Florida

are summarized in Chapter 2. This section considers the climatic conditions, geology, physical and hydrodynamic features, and water quality found in developed areas along the coast of Florida. The regional and seasonal variability of these features is an important consideration in determining the types of coastal areas which would be relatively acceptable for canal development.

Canal design in Florida is guided by legislation, the economics of development, and environmental considerations. To some extent, it is also influenced by past design practices, some of which, both good and bad, have become widely accepted in practive through continued use. In Chapter 3 these factors are outlined and discussed as they pertain to present day design objectives and limitations.

Chapter 4 describes the initial planning for canal design in terms of criteria, guidelines, and constraints. Design objectives are defined as the qualitative guidelines under which the canal designer and the developer cooperate to produce a set of quantitative design criteria. These criteria are established by considering various suggested guidelines that have been developed from consideration of problems which have been identified with canals in the past.

The first step in planning a canal development is an evaluation of the characteristics of the site. Chapter 5 distinguishes between fixed site characteristics and alterable site characteristics, and describes sources of published information on regional data which are applicable to this problem. Then, preliminary site investigations are described in terms of their objectives and the monitoring and sampling problem. A general discussion of the measurement requirements for tidal canals completes this Chapter.

The results of the field work guided the development of the basic features of the canal design model. Chapter 6 begins with a description of desirable specifications for canal instrumentation, and then describes the instrumentation and support equipment used by the Hydraulic Laboratory in the canal design research project. The reduction and presentation of field data follows. The field procedures carried out by the author for the Hydraulic Laboratory are next described, with some observations on the results. A specific analysis of the results of the comprehensive dye dispersion experiments, however, has been incorporated into a discussion of the calibration of the numerical model at the end of Chapter 7.

The development and features of the numerical model of mass-transport in low energy canal networks, a research project conducted by R. Walton in conjunction with the project described herein, are summarized in Chapter 7. The characteristics of one- and three-dimensional mathematical models, the development of the three-dimensional numerical model, and its stability are covered in some detail, but not completely, as the work is not that of the author. The chapter concludes with a discussion of the verification of the model, with some explanation by the author of the field results as they pertain to this aspect of the research.

In Chapter 8 an overall design process is described. This process begins with the development of quantitative design constraints and criteria, specific examples of which are given. Design elements are described, and some general guidelines as to their use in synthesizing canal networks are tabulated. The variability of both the one- and three-dimensional numerical models is shown for variations in

geometry and boundary conditions, which gives some insight into the operation and design of tidal canals and canal networks. The final part of Chapter 8 describes some of the features of basic canal design elements: the comb-structured network, the "lake," bends, and a simple system with two tidal entrances and a nodal point. These are simulated using the CANNET3D model, and the results are compared in terms of relative flushing time.

An example consisting of a hypothetical "existing" canal network and some tests to find its optimal depth and flushing characteristics are developed in Chapter 9. General simulation objectives are described as well as several methods for quantifying the flushing characteristics of a given network. The resulting concentration profiles after fifty tidal cycles, under no-wind and mild-wind conditions, and with and without additional tidal prism, are compared. The chapter concludes with a discussion of the relative effect of constant and variable winds.

Chapter 10 is a summary of the project, together with conclusions and recommendations.

Table 1.1 - Inventory of Large-Scale Developments in Florida.

Name	Location	Developer	Acreage	Res units	Ultimate pop	Current pop
North Golden Gate	Collier	GAC Corporation	2 500	6 000	26 000	
Cape Coral	Lee	GAC Corporation	4 100	100 000	300 000	16 371
Golden Gate Estates	Collier	GAC Corporation	112 000b	0	0	
River Ranch Shores	Osceola	GAC Corporation	4 [illegible]	15 000	40 000	
River Ranch Acres	Osceola	GAC Corporation	49 [illegible]b	0	0	
Remuda Ranch Grants	Collier	GAC Corporation	60 000b	0	0	
Barefoot Bay	Indian River	GAC Corporation	1 100	5 000	11 000	0
Ocala Springs	Marion	GAC Corporation	4 [illegible]0	12 000	40 000	
Poinciana	Osceola Polk	GAC Corporation	4[illegible] 100	70 000	250 000	0
Golden Gate	Collier	GAC Corporation	[illegible] 000	6 000	26 000	
Port Charlotte	Charlotte Sarasota	General Development	100 840	140 [illegible]	500 000e	28 500
Port LaBelle	Hendry Glades	General Development	31 530	50 000	130 000e	10
Port St Lucie	St Lucie	General Development	44 480	81 100	220 000e	9 500
Port Malabar	Brevard	General Development	43 500	70 449	185 000e	6 250
Vero Bch Shores Hghlds	Indian River (2)	General Development	5 586	9 297	25 000e	1 400
Sebastian Highlands	Brevard	General Development	1 556	2 324	6 000e	658
Julington Creek	St Johns	General Development	4 958	13 245	45 000e	650
		General Development	1[illegible] 508	0	0	
Deltona	Volusia	Deltona Corporation	1[illegible] 700	36 000	104 400	10 866
Marco Island	Collier	Deltona Corporation	9 100	25 800	75 500	5 330
Spring Hill	Hernando	Deltona Corporation	16 400	34 500	100 000	4 600
Citrus Springs	Citrus	Deltona Corporation	15 000	33 000	95 600	1,085
Sunny Hills	Washington	Deltona Corporation	17 100	34 000	98 600	175
St Augustine Shores	St Johns	Deltona Corporation	2 600	6 000	12 000	0
Pine Ridge Estates	Citrus	Deltona Corporation	9 584	5 000	14 [illegible]	0
Marion Oaks	Marion	Deltona Corporation	15 000	33 280	96 500	0
Royal Palm Bch Village	Palm Beach	Royal P B Colony Inc	4 000	14 900	[illegible]	1 068
Royal Highlands	Hernando	Royal P B Colony Inc	8 000	14 000	60 000	
Holiday by the Sea	Santa Rosa	Royal P B Colony Inc	5 000	5 900	15 000	
Royal Trails	Lake	Royal P B Colony Inc	11 000	5 800	16 500	
Cypress Springs	Orange	Gulfstream Land & Devmt	2 900b	0	0	
Switzerland Forests	St Johns	Gulfstream Land & Devmt	1 800b	0	0	
Gulfstream Plantation	Brevard	Gulfstream Land & Devmt	5 400	13 300e	13 300e	
Venice Gardens	Sarasota	Gulfstream Land & Devmt	1 100	6 800e	19 000e	
Winter Springs	Seminole (PUD)	Gulfstream Land & Devmt	5 400	7 700e	21 600e	
Argile Forests	Duval Clay	Gulfstream Land & Devmt	8 000	17 900e	49 300e	
Coral Springs	Brevard	Coral Ridge Properties c	13 000	53 000	130 000	6 715
Spring Lake	Highlands	Coral Ridge Properties c	6 400	18 000	50 700e	
Naples by the Sea	Collier	Coral Ridge Properties c	2 100	4 600e	13 000e	
Sandestin	Walton	Evans & Mitchell c	2 000	6 000	18 000	
Halifax Plantation	Flagler	Evans & Mitchell c	5 600	16 800	50 000	
Perdido Bay C C Estate	Escambia	Cavanaugh Comm Corp	2 400	15 805	36 232	365
Rotonda West	Charlotte	Cavanaugh Comm Corp	26 000	65 688	164 000	520
Orange Wood	Orange	Florida Land Company c	4 500	17 657	48 000	100
Winter Springs (N Orlando)	Seminole PUD 550 ac	Florida Land Company c	2 250	1 664	4 500	1 161
Killearn Estates	Leon	Killearn Properties c	3 485	5 630	15,500	2 600
Killearn Lakes	Leon	Killearn Properties	3 982	9 154	25 200	175
San Carlos	Lee	American Intl Land Corp	2 754	7 010	17 525	500
San Carlos Estates	Lee	American Intl Land Corp	1,160 + b	0	0	
San Carlos West 800	Lee	American Intl Land Corp	720 + b	0	0	
Peace River West	Desoto	American Intl Land Corp	4 000 + b	0	0	
Peace River East	Desoto	American Intl Land Corp	2 100 + b	0	0	
Hidden Lakes Ranch	Glades	American Intl Land Corp	3 545 + b	0	0	
Lehigh Acres	Lee	Lehigh Acres Dev Inc	60 000	28 500e	80 000	4 394
Lake Buena Vista	Orange	WED Enterprises	4 580 +	9 650	27 000	18
Palm Coast	St Johns Flagler	ITT	100 000	250 000e	750 000	600
Crescent Estates	Pasco	Hutland Investment Corp	2 821	20 000	52 500	50
Suhara	Pasco	Hutland Investment Corp	2 500	15,260	35 000	0
Tomoka Springs	Volusia	Recreational Systems Inc	2 746	1 000	4 015	0
Placid Lakes	Highlands	Lake Placid Holding Co	7 500	15 000	35 000	600
Wellington	Palm Beach	Breakwater Hsng Corp	7 400	14 800	37 000	0
Amelia Island Plantation	Nassau	Sea Pines Company	3 700	2 700	4 000	10
Compass Lake	Jackson	Compass Lake Dev Corp	12 000b	0	0	
Avon Park Estates	Highlands	Avon Park Estates Corp	3 500b	0	0	
Rolling Hills	Marion	MRI Properties Inc	5 400b	0	0	
Miami Lakes	Dade	Sergeant	8 000e	25 000		
North Palm Beach	Palm Beach	(Ross Brothers) c	9 600	8 900e	27 000	10 923
Palm Beach Lakes	Palm Beach	Perini Corporation c	7 000	25 000e	70 000	
Palm Beach Gardens	Palm Beach	M Arthur c	6 100	25 000e	70 000	7 620
Port Port Colony	Hillsborough	Intervest Inc c	2 565	9 149	30 674	0
(Fletcher)	St Johns	Fletcher Prop Inc c	5 400b	0	0	

b Held in undeveloped acreage no plans for development currently
e Estimated by DSP
c Data from unverified sources

Source: Carter, 1974, pp. 32 & 33.

Table 1.2 - Criteria for Canal Design Without Consideration of the Environment (after Snyder, R.M., *Residential Canals and Environment*, Snyder Oceanography Services, Jupiter, Florida, 1976).

GOAL	INCIDENTAL FACTORS	ENGINEERING CRITERIA	RESULTING DEVELOPMENT	ENVIRONMENTAL IMPACT
	NO CENTRAL SEWAGE TREATMENT		SEPTIC TANK SEEPAGE	DOMESTIC WASTE POLLUTION
		NAVIGABLE DEPTHS TO SHORELINE	ELIMINATION OF SHALLOWS	NO NATURAL WATER TREATMENT
RESIDENTIAL UTILIZATION OF WATERFRONT		MAXIMIZE FRONT FOOTAGE PER ACRE	STRAIGHT LINE CANALS WITH RIGHT ANGLE BENDS	POOR MIXING (LOW DISPERSION)
		INCREASED ELEVATION FOR FOUNDATIONS	DEEP NARROW CANALS	
		MINIMUM LOSS OF PROPERTY TO WATER AREA	VERTICAL BULKHEADS	LIMITED HABITAT
		RAPID DRAINAGE OF RAINFALL	DIRECT STORM WATER DISCHARGE	STORMWATER POLLUTION
		SIMPLIFIED SURVEYING & CONSTRUCTION METHODS	WATER TABLE DRAWDOWN FROM LOCAL WELLS	SALTWATER INTRUSION
	NO OFFSITE WATER SUPPLY			----------
				DEGRADATION OF WATER QUALITY AND BIOLOGICAL HABITAT

SNYDER OCEANOGRAPHY SERVICES

EARLY CANAL DESIGN

Figure 1.1 - Example of Bayfill Development in Florida.

Figure 1.2 - Example of Intertidal Development in Florida.

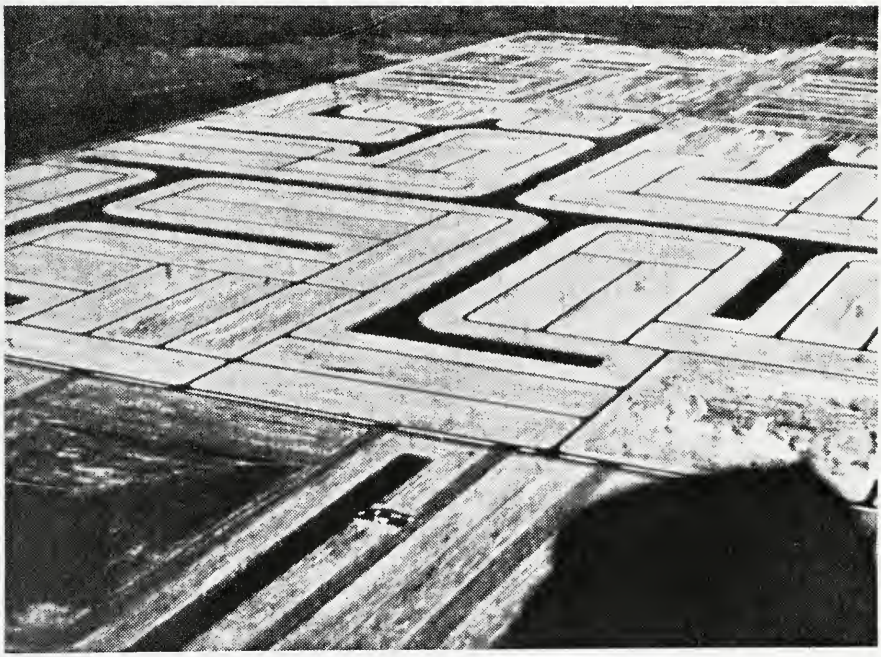
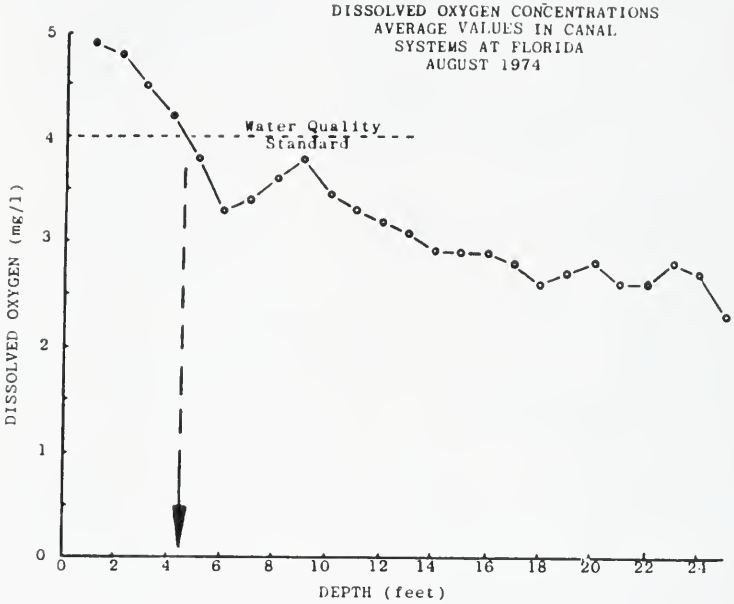


Figure 1.3 - Example of Inland Canal Development in Florida.



DISSOLVED OXYGEN STATISTICS
CANAL SYSTEMS AT FLORIDA
AUGUST 1974

Depth	No. of Observations	Avg DO	Range	% > 4.0	% ≤ 3.0	95% Confidence	Interval	Std Deviation
1	134	4.9	8.7-1.4	84	18	2.7	7.1	1.1
2	134	4.8	7.8-1.5	80	20	2.6	7.0	1.1
3	134	4.5	8.8-1.4	71	30	2.2	6.8	1.1
4	134	4.2	8.8-0.8	64	38	1.8	6.8	1.3
5	134	3.8	8.8-.0	55	45	0.3	7.3	1.8
6	134	3.3	7.0-.0	47	53	0	7.4	2.1
7	135	3.4	7.0-.0	46	54	0	7.5	2.1
8	99	3.6	7.2-.0	44	58	0	7.5	2.0
9	67	3.9	8.0-.0	48	54	0	7.4	1.8
10	40	3.4	7.1-.0	40	60	0.4	6.4	1.5
11	33	3.3	5.9-1.0	33	67	1.1	5.5	1.0
12	30	3.3	5.1-1.0	23	77	1.1	5.5	1.0
13	28	3.1	8.0-1.1	14	88	1.1	5.1	1.0
14	24	2.8	4.3-1.1	8	92	0.9	4.9	1.0
15	34	2.8	4.2-1.1	8	93	1.0	4.8	0.8
16	22	2.9	4.2-0.8	9	91	0.9	4.8	1.0
17	17	3.8	4.0-0.7	5	94	0.7	4.9	1.0
18	15	2.8	4.0-0.7	7	93	0.5	4.7	1.0
19	14	2.7	3.8-1.2	0	100	1.0	4.4	0.8
20	11	2.8	3.8-0.8	0	100	1.3	4.4	0.7
21	9	2.8	3.6-0.8	0	100	0.8	4.8	0.8
22	8	2.8	3.8-1.3	0	100	0.9	4.3	0.9
23	8	2.8	3.4-2.3	0	100	1.8	3.8	0.4
24	2	2.2	2.8-2.4	0	100	-	-	-
25	1	2.5	2.5	0	100	-	-	-

Figure 1.4 - Average Values of Dissolved Oxygen Concentrations in Canal Systems in Florida, August 1974 (Source: EPA, 1975b, p. 12).

CHAPTER 2

FEATURES OF RESIDENTIAL CANAL SYSTEMS

This section combines a literature review with a general, qualitative explanation of the hydrodynamics and water quality of tidal residential canal systems along the south Atlantic and Gulf Coasts. The examples and data are taken somewhat at random from the existing information for Floridian canals. It should be noted that canals primarily constructed for navigation, flood control, irrigation, mosquito control, and other such purposes are not specifically covered, as the problem of *residential* canal design is being addressed. Of course, the physical principles governing the operation of tidal canals do not vary with their purpose, but rather with their form and location, so that many of the concepts presented here can be applied in other contexts.

2.1 Governing Features of Tidal Canal Systems

The hydrodynamics of tidal canals are governed by the combined effects of canal system layout, channel geometry, climate, tidal range and period, wind, the saltwater/freshwater balance, water circulation, and certain geological features. Water quality is dependent on the hydrodynamics, but does not affect the flow in the canal, and thus can be used as an indicator of the effectiveness of the circulation in mixing and flushing pollutants from the canal system.

A given canal system is part of the surrounding ecosystem, which has terrestrial, marine, and atmospheric components. Thus, there are

many interactions between the canal system and the other parts of its natural surroundings. However, the mass-transport characteristics of a canal system may be effectively described in terms of a pollutant with a first-order decay, with the influence of the ecosystem being limited to the physical forcing functions (tides, winds, and salinity and temperature gradients), the effect of vegetation on runoff, and the interaction of water currents with bank and bottom materials. Since the basis for the canal model is the hydrodynamic laws and the mass-transport of a substance without interactions, the numerical model could be adapted at a later time for the evaluation of water quality dynamics.

2.2 The South Atlantic and Gulf Coastal Zone

Lying between latitudes 24°30' and 31° North, and longitudes 80° and 87°30' West, Florida has 1,266 miles of general coastline or over 9,000 miles of detailed coastline (including all indentations and the shores of islands). Of the general coastline, 593 miles border the Alantic Ocean and the remaining 673 miles lie on Florida Bay and the Gulf of Mexico [Corps of Engineers, 1971, p. d1; Carter, 1974, p. 1]. The coastal zone as defined by the Florida Coastal Coordinating Council [1971] is shown in Figure 2.1. The definition of the coastal zone is based on population density rather than on the topography of the area. It is a political definition, not a physical one.

The Atlantic shoreline of Florida consists of a series of sea islands and barrier islands separated from the mainland by the continuous Intracoastal Waterway. North of Jacksonville the sea islands border salt marshes, while to the south as far as Miami the barrier island chain, which is broken by 14 of the 57 inlets on the Florida

coast, is backed by low tidal marsh or lagoons. These barrier islands vary considerably in their dimensions and the degree of development, from wide flat beaches with 10 to 20 ft high dunes to narrow sand strips fronting seawalls. Some of the inlets open into extensive estuaries, while others formed by storms breaching the barrier islands have no associated estuary or bay.

The Florida Keys may be divided into three distinct groups. The long, narrow keys occurring at the northern end of the chain are coral, the irregularly shaped Keys in the center portion are Miami oolite (limestone), and the lower Keys consist of oolite interspersed with patches of mangrove. The ocean bottom to the east of the Keys is a nearly flat sheet of limestone extending from one-half to one mile offshore. The side facing Florida Bay is similar, with scattered mangrove located in the bay to the west of the upper Keys.

Most of the Gulf shoreline from Florida Bay northward to Anclote Key (at the Pinellas/Pasco County line) consists of offshore barrier islands, beginning with the Ten Thousand Islands at the south end of the peninsula. The southern part of this coastline is characterized by extensive networks of tidal creeks and mangrove swamps. Shallow tidal lagoons lie between the offshore islands and the mainland. These barrier islands are separated by shallow, natural passes which in some instances have been extensively improved for navigation.

North from Anclote Key to Apalachee Bay the coast completely changes character. The barrier beaches are replaced by low, flat tidal marshes, and the slope of the bottom offshore is very slight. Beginning with the sand spits at the west end of Apalachee Bay the coastline again becomes predominantly a series of beaches on the mainland. These

beaches are generally wide, with 10 to 15 ft sand dunes. [Information primarily from Corps of Engineers, 1971, p. d1].

2.3 Climate

Climate is an important consideration in the design of a canal system because wind, precipitation and storms affect the circulation of water in the canals. In addition, cloud cover affects the air and water temperature and the productivity of the canal biota.

The climate of Florida is categorized as transitional between temperate and subtropical in the extreme northern interior of the state, and tropical in the Florida Keys. The climate is controlled and moderated primarily by the latitude, proximity to the Atlantic Ocean and the Gulf of Mexico, and the 4,400 square mile surface area of inland lakes. The summer season throughout the state is relatively long, warm and often humid, while the winters are comparatively mild due to the southern latitude and warm coastal waters. The Gulf Stream has a warming effect on the east coast because winds prevail from the ocean.

Florida has abundant rainfall. Most localities receive over 50 inches per year, except the Florida Keys which average about 40 inches per year. A typical year can be subdivided into two "rainfall seasons" throughout all of the state except the Panhandle. On the peninsula portion more than one-half of the yearly precipitation usually falls from June through September, which climatologists call the "rainy" season. The other eight months comprise a relatively dry season. In the Panhandle a secondary rainfall maximum occurs in late winter and early spring. In addition, the distribution of rainfall with a given year is quite uneven. During the rainy season the probability that rain will fall on any particular day is about 50 percent, while during the

remainder of the year rain may be expected to fall on one or two days during the week.

The seasonal distribution of rainfall varies from north to south. On the peninsula this distribution is dominated by a large amount of summer rainfall and the rather abrupt start and end of the summer rainy season. In the Panhandle there are two times of high rainfall, one in late winter or early spring and the other during the summer. The time of lowest rainfall occurs in October, and secondary low quantities occur in April or May.

Local showers or thundershowers are common in summer. Many localities average more than eighty of these storms in a year, while some experience more than a hundred. These showers are generally quite heavy, up to three inches in two hours and ten inches in twenty-four hours. The more severe storms are occasionally accompanied by high, damaging winds. However, typical summer storms are of relatively short duration and even in the rainy season rainfall usually occurs less than 10 percent of the time.

While Florida normally has substantial rainfall during a year, portions of the state have also experienced severe droughts. These dry periods, which may last longer than a month, occur even during the normal time of the rainy season, resulting in excessively low water levels in reservoirs and aquifers.

Over the southern part of the peninsula winds prevail from the southeast and east. In the remaining part of the state they are somewhat erratic, but predominately north in winter and south in summer. The months with the highest winds are March and April, and high local winds of short duration are generally associated with thunderstorms in

summer and with cold fronts in other seasons. Tornados have occurred at all times of the year, but they usually do not cause extensive damage.

Tropical storms, and particular hurricanes, are another matter. A tropical storm is any storm that produces high winds (above 34 knots) and therefore has destructive potential. A hurricane is a tropical storm with maximum winds of 64 knots or more. From 1885 through 1977, 67 tropical storms and 86 hurricanes have entered or significantly affected Florida [Environmental Data Service, (monthly) to 1977]. The average number of tropical storms is 1.7 per year, with a variation of from none to 5 in any year. Florida has not experienced more than 3 consecutive years without a tropical storm, nor more than 5 consecutive years without a major hurricane.

The probability that hurricane force winds will impact a particular Floridian city in any year is summarized in Table 2.1. This probability varies from a low of one in one-hundred at Jacksonville to a high of one in seven at Key West and Miami. In the ninety-three years of record, only ten or eleven hurricanes have passed inland on the west coast in the area from Cedar Key to Fort Myers. Along the coast from Jacksonville to St. Augustine the first recorded hurricane was experienced in 1964.

The probability of experiencing a tropical storm increases as the hurricane season develops. In August and early September hurricanes normally approach from the east or southeast, while in late September through October hurricanes generally approach from the western Carribbean into the Gulf of Mexico. Wind gusts of up to 155 mph have been recorded (accuracy unknown), but anemometers do not usually survive winds in this range. Wind speeds up to 200 to 250 mph have been calculated based on the extent of damage in the most intense recorded

hurricane. It is estimated that sustained winds of over 150 mph are experienced in Florida approximtely every seven years. Very heavy rainfall occurs within tropical storms, over 20 inches in 24 hours having been occasionally measured. The average hurricane rainfall, however, usually does not exceed 8 inches in 24 hours.

The sky is overcast about one-third of the possible sunlight hours during a year, ranging from a value of less than forty percent in December and January to less than thirty percent in April and May. In general, hours of sunshine in the southern part of the state exceed that for the northern part. The contrast between the amount of daily sunlight in New York and Miami is substantial. In December, the number of sunlight hours in Miami averages sixty-six percent and in New York fifty-one percent, but Miami receives an average of 317 langleys on a horizontal surface while New York receives an average of only 116 langleys [Bradley, 1974, pp. 45-70].

2.4 Physical Features of Floridian Canal Systems

2.4.1 Types of Canals

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 that do take advantage of the existing topography will encounter the least resistance to development, both physical and political.

Christensen [in Christensen and Snyder, 1978] has classified existing straight canals into five major groups, and added borrow pits as a sixth category for consideration of flushing characteristics:

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

These six classifications are diagrammed in Figure 2.2. Complex networks may be obtained by combining one or more of these groups, with or without curves. A *flow-through* canal (or canal reach) is one that maintains a flow of water at its two open boundaries, in contrast to a *simple dead-end* canal, which has only one boundary open to flow. The latter is often associated with poor water quality because velocities become small near dead-ends, sediments fall out of suspension more easily, and surface debris tends to accumulate in the dead-ends unless a favorable steady wind is able to carry this debris out. The flow in the vicinity of a dead-end is complicated by upward or downward water movement when two- and three-layer wind-induced flows occur in the channel.

A *higher-order* finger canal network 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 network with many relatively short, parallel, closely spaced dead-end branches or fingers is a *comb-structured* network. These fingers may be straight or curved, as shown in Figure 1.1. A canal with

a *lagoon* or basin at one end, 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 the channels connecting a basin to the receiving waterbody will increase as the basin area is increased, which in turn will increase mixing and flushing in the connecting channels. The location and sizes of lagoons or boat basins are therefore an important design element in a canal network.

2.4.2 Canal Banks

In the past, canal channels have often been constructed with vertical bulkheads, not only to hold the fill from the channels but to maximize the lot size and number of lots within the available development area. Vertical bulkheads (shown in Figure 2.3) are convenient for mooring boats, especially deep-draft vessels. Although they provide an adequate environment for certain sessile organisms, they have a number of serious disadvantages. One of the most serious is the susceptibility of the bulkhead, if not designed properly, to erosion at the toe due to boat wakes and to runoff and undermining on the landward side. They also reflect boat wakes back into the channel, which can cause dangerous navigation conditions.

The more desirable trapezoidal channel shape, which is similar to a natural channel with sloping banks (Figure 2.4) is not often found in residential canal developments. Substantially greater widths are required to permit a navigable section along the centerline, which utilizes land area that would otherwise be available for housing lots. The banks may either be cut directly into the upland soils at a slope

that will be stable for the design channel velocities, or may be established at some arbitrary slope and riprapped. If constructed on native soil, they should be vegetated to provide soil stability and to dissipate the energy in boat wakes.

2.4.3 Tidal Characteristics

The relatively small range of tides around the coast of Florida, and the resultant low level of energy available from this source for mixing and flushing of pollutants, has already been mentioned. Along the coast of Florida all of the three major types of tides are encountered, as shown in Figure 2.5. Examples of each type are given in Figure 2.6. Mixed tides exhibit alternate high and low values of substantially different elevations, which can have the effect of decreasing the flushing action on alternate tidal cycles.

At the beginning of the canal research project it was not known whether the slope of the water surface due to the tidal wave would be significant or not, although it was suspected that it would be quite small. Attempts to measure the slope with tide gauges showed it to be of the order 10^{-5} to 10^{-6} or less. Therefore, a comparison of tidal elevations at the end of a 5,000 ft straight canal was made using two different one-dimensional numerical models, one including the momentum equation (which takes this effect into account) and one that assumes that the water surface is always horizontal [Walton, in Morris, Walton and Christensen, 1978]. The water surface elevations and velocities compared within 2 percent, which was concluded to be sufficient justification for ignoring the surface slope considering that the combined measurement accuracy for tidal elevations, current velocities, dye studies, and predictions using numerical models would be significantly

lower. The physical explanation that justifies the horizontal water surface assumption is that the tidal elevation changes relatively slowly over a tidal half period, on the order of 3 ft/6.21 hour or 0.008 fpm.

2.4.4 Tidal Energy

Part of the energy brought into the canal system by the tide is expended in overcoming frictional resistance and in mixing, while the remainder is stored as potential energy for the next half tidal cycle. The total potential energy stored in the tidal prism in a given canal system after a flood tide is

$$E = \gamma V a \tag{2.1}$$

where

E = potential energy, (FL)

γ = unit weight of water, (F/L^3)

V = volume of tidal prism, (L^3)

a = elevation of centroid of tidal prism

= amplitude of tide, (L)

The total power, or rate of energy storage, in horsepower is found by dividing by the half-tidal period and the conversion factor for lb-ft/sec to hp

$$P = \frac{\gamma V a}{\frac{T}{2} \times 3600 \times 550} \tag{2.2}$$

$$= \frac{2.64 \times A_s \times 2a \times a}{12.42 \times 3600 \times 550}$$

$$= 1.041 \times 10^{-5} \times a^2 A_s \text{ (hp)}$$

where the units of a and A_s are ft and ft^2 respectively and

P = power available from tidal prism, (FL/T)

T = tidal period, (sec)

A_s = surface area of canal, (L^2)

Thus, the power available for flushing on the ebb flow at any location in the canal system is given by Equation (2.2), using the mean tide surface area upstream (in the direction away from the tidal entrance) of that location for A_s. For example, in one large system studied by the Hydraulic Laboratory, the 57 Acres canal system in Palm Beach County, on the intracoastal waterway, the mean tidal amplitude is approximately 1.25 ft, the total surface area is about 4,300,000 ft^2 (98.7 acres), and the power available is 70 hp. The available power from stored tidal energy in the 1700 to 2400 ft long west coast finger canals studied by the Hydraulic Laboratory varies between 1 and 2 hp. This energy is supplemented by energy from the wind and salinity gradients for flushing.

As the tidal wave travels into the canal system it will progressively lose some of its energy and it will take a finite time to reach locations within the canal system. Measurements by the Hydraulic Laboratory in the 57 Acres canal system, using tide gauges, showed a tidal elevation difference of 0.04 ft and a lag of 0.8 hours at slack tide over a distance of 7780 ft from one entrance, and a tidal elevation difference of 0.01 ft and a lag of 0.6 hours at slack tide over a distance of 9695 ft. The calculated water surface slopes at maximum tidal discharge over these distances were 5×10^{-5} and 3×10^{-5} respectively [Walton, et al, 1975b, p. 59].

Since the tide loses energy as it progresses up a dead-end canal system, there is, at least theoretically, a limited distance over which it will be effective in mixing. This may be called the tidal excursion distance, x', which can be theoretically calculated if it is assumed that there is no mixing of the flood tide with the resident water. Conceptually, under these circumstances the flood tide pushes the resident water toward the dead end and the tidal prism, awL, fills the entrance end of the canal to a distance x' or

$$(d_o + a)\, wx' = 2awL$$

$$x' = \frac{2a}{d_o + a} L \tag{2.3}$$

where

d_o = mean (mid-tide) depth, (L)

L = length of canal, (L)

By this concept the maximum possible value of the excursion distance is the length of the canal, when a equals d_o. In an eight-ft deep canal, with tidal amplitude equal to 1, the excursion distance by this formula would be 0.22 L.

2.4.5 Secondary Currents

It is known that secondary currents occur in straight channels as a result of turbulent velocity fluctuations and the normal stress produced by turbulence [Schlichting, 1968, p. 576; Ikeda and Kikkawa, 1976]. Velocity measurements taken by the Hydraulic Laboratory in a canal several miles up the Loxahatchee River indicate that even in a straight, 2200 ft dead-end canal the flow is not at all uniform in any of the coordinate directions (Figure 2.7). Some lateral nonuniformity in the depth, and transverse wind components, contribute to the formation and

maintenance of the secondary flows that are observed there. Further evidence of the presence of secondary flows is a series of evenly-spaced, dome-shaped shoals along the banks of the canal that are uncovered at low tide.

Secondary currents of much greater significance from the viewpoint of mixing also occur in bends, producing velocities an order of magnitude greater than those produced in straight reaches. These currents are formed by the effect of the centrifugal force exerted on the water particles due to higher velocities at the surface than at the bed. This results in a superelevation of the water surface on the outside of the bend and lower elevation on the inside of the bend, and secondary current flowing downward on the outside and upward on the inside. The superposition of the secondary flow on the primary water movement results in a helical water particle path, as shown in Figure 2.8. Secondary flows are an important design element because they are instrumental in vertical mixing.

2.4.6 Dispersion Coefficients, Flushing Time, and Models

The ability of a canal system to flush pollutants to the receiving waterbody, and thereby to maintain water quality in the canals, is fundamentally related to the way in which incoming natural energy is distributed and used in the canal system. Principally, energy is brought into a canal system by the tides and the wind. A secondary source is freshwater inflow and the resulting salinity gradients. In describing the spatial and temporal changes of energy it is convenient to employ descriptions of the circulation of the water and the distribution of pollutant concentration gradients, to which, in turn, the movement and spreading of pollutants can be directly related.

The movement of the water convects pollutants, while concentration gradients cause the pollutants to distribute themselves in the direction that will tend to reduce these gradients. In turbulent flow, which is the type of flow almost always found in open channels, the local circulation is characterized by the presence of velocity fluctuations in all directions, and transverse and vertical eddies. Together these fluctuations and eddies cause turbulent mixing.

The "flushing time" of a canal may be considered as the time required to replace the polluted water in the canal with unpolluted water [Bowden, 1967, p. 19]. The "residence time" has been generally defined as the time that a particle of water remains in the canal before it is exchanged with another particle from outside the system. The following expression for the flushing time, T_F, or the mean residence time, T_R, can be derived by ignoring the dynamics of the circulation and the details of the flushing process and assuming that steady state conditions prevail (i.e., that the pollutant is removed at the same rate as it is introduced).

$$T_F = T_R = \frac{V}{Q_i} \tag{2.4}$$

where

V = mean volume of polluted water to be removed, (L^3)

Q_i = mean rate of removal of polluted water, (L^3/T)

The "mixing half-life" of the canal would be given by $T_F/2$. Equation (2.4) has been derived by neglecting the mixing of incoming polluted water with the resident canal water and ignoring any density-induced net circulation (stratified flow) or wind influence. Because of the assumptions involved, flushing and residence times calculated in this way are

meaningless for tidal canals.

Many more sophisticated "flushing time" models have been devised, most of which involve assumptions of complete mixing over at least some part of the waterbody. For example, in "tidal prism" methods it is assumed that all of the water entering on the flood tide becomes completely mixed with the resident water in the canal, while in "segmented" models [Ketchum, 1951] complete mixing is assumed to take place successively in segments with lengths determined by the tidal excursion. Still another variation is to introduce an "exchange ratio," which effectively reduces the volume over which mixing is assumed to take place. While such models attempt to take the variation of salinity into account, and will in some cases provide results to the correct order of magnitude, there are still too many assumptions involved to give a reliable estimate of flushing time for tidal canals.

Longitudinal dispersion is one of the processes by which a mass of some dispersant, e.g. a pollutant, is spread out, mixed, and thus diluted in a flow of water, the others being molecular diffusion and turbulent diffusion. The term "dispersion" applies to spreading that is controlled by spatial velocity gradients, in contrast to "diffusion" which is spreading caused by random temporal fluctuations. Thus, dispersion is caused by nonuniform transverse and vertical velocity profiles. In turbulent flow in natural waterways it has been found that the velocity gradients are far more important in determining the dispersion rate in a given canal than either molecular diffusion or turbulent diffusion, which are essentially random and therefore not occurring in any particular direction.

In the derivation of the one-dimensional convective-dispersion equation for turbulent flow

$$\frac{\partial}{\partial t}(Ac) + \frac{\partial}{\partial x}(Auc) = \frac{\partial}{\partial x}(AE_\ell \frac{\partial c}{\partial x}) + Ar_p \tag{2.5}$$

$c(x,t)$ = concentration, (dimensionless)

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

$u(x,t)$ = spatial mean velocity, (L/T)

E_ℓ = longitudinal dispersion coefficient, (dimensionless)

r_p = rate of production or removal of pollutant mass, $(1/T)$

it is assumed that the dispersion process can be approximately described by a one-dimensional Fickian-type diffusion equation. Under this assumption, in theory, the variance of the concentration distribution of a conservative dispersant in steady flow should increase linearly with time. For uniform steady flow a variety of analytic solutions are available for the concentration distribution as a function of location, time, and the characteristics of the source of the dispersant. For example, the solution for an instantaneous plane source of dispersant uniformly distributed over the cross-section of a channel is given by

$$c(x,t) = \frac{M}{\rho A(4\pi E_\ell t)^{1/2}} \exp - [\frac{(X - \bar{U}t)^2}{4E_\ell t}] \tag{2.6}$$

where

M = mass of pollutant injected, [m]

ρ = density of solution, (M/L^3)

X = distance from point of injection of dispersant, (L)

t = elasped time since injection, (T)

$\bar{U}$ = mean (steady, uniform,) velocity of low from injection point to sampling point, (L/T)

This equation indicates that the pollutant has a Gaussian distribution in the x-direction for all time and the peak concentration is always at $x = \bar{U}t$, and decreases with time according to

$$c_{max} = \frac{M}{\rho A \sqrt{4\pi E_\ell t}} \tag{2.7}$$

The centroid of the dispersing cloud is also at $x = \bar{U}t$, and the variance of the distribution of concentration is given by $\sigma^2 = 2\ E_\ell t$ [Holley and Harleman, 1965, pp. 59-60].

This solution, and others similar to it for continuous sources, in one-, two-, or three-dimensions, form the basis for the design and analysis of a great many dispersion experiments. The objective of a dispersion experiment is to determine a value for the coefficient E_ℓ from measurements of tracer distributions. One form of the dispersion coefficient E_ℓ is [Taylor, 1954; Elder, 1959]

$$E_\ell = KRu^* \tag{2.8}$$

where

K = dimensionless dispersion coefficient

R = hydraulic radius (L)

u^* = bed shear velocity, (L/T)

The dimensionless dispersion coefficent 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 correspond to irregular curved and meandering systems.

Besides the restriction that the mean velocity of flow must be approximately constant over the period of a dispersion experiment, there is a further limitation described by Fischer [1967b, pp. 192 and 207]. For dispersion to be described by the diffusion equation (i.e. the Fickian equation) it is necessary that the motion of each tracer particle not be dependent on its initial velocity. The period during which this is true is called the "Taylor" period. The initial period after injection of the tracer material is called the "convective" period, and Fisher found that the criterion for use of the one-dimensional convective-dispersion equation and a Taylor-type dispersion coefficient (Equation 2.8) is that the distance downstream from the point of tracer injection to the sampling point should be [Fischer, 1967b, p. 213]

$$L > 1.8 \frac{\ell^2}{R} \frac{\bar{U}}{u^*} \tag{2.9}$$

where

L = distance between injection point and sampling point, (L)

ℓ = a characteristic length of the cross-section

= distance from the point of maximum surface velocity to the most distant bank, (L)

For example, in an 80 ft wide canal, 8 ft deep, with a uniform transverse velocity distribution, a mean flow velocity of 0.2 fps, and u^* typically 0.05 fps, L is 1450 ft. Thus, if the canal in which the study is to be conducted is 1500 ft or less in length, the tracer must be

well-mixed across the channel near the dead-end for the experimental results to be valid.

One-dimensional convective-dispersion models have been available since the late 1960's and have been applied to the simulation of pollutant dispersion in rivers and in estuaries. While there are many reviews of the capabilities of a variety of numerical models (for example, see Grimsrud et al [1976] and Lombardo [1973]), the comments by EPA, [1975b, pp. 207-209] on the process of selecting a model for application to flushing in tidal canals are most pertinent here. EPA began by reviewing the Feigner and Harris [1970] version of Water Resources Engineers' San Francisco Bay model, called the Dynamic Estuary Model (DEM). This had been incorporated into the Stormwater Management Model (SWMM), University of Florida 1973 version [Metcalf and Eddy, 1971], which was used for EPA's initial canal flushing simulations. The fact that diffusion and dispersion were not incorporated into these models finally led EPA to choose the Columbia River Model (CRM) version of the original Water Resources Engineers model [Callaway et al, 1970] for simulating flushing in canals, but since this is only a one-dimensional model it could not simulate canal flushing properly [Walton, in Morris, Walton, and Christensen, 1978, p. 342-343].

The effect of mixing in numerical models is expressed in terms of various coefficients. The mixing coefficients associated with turbulence, and obtained by averaging turbulent velocity fluctuations over time, are called turbulent diffusion, turbulent diffusivity, or eddy diffusivity coefficients, usually expressed in each of the coordinate directions. The mixing coefficients evaluted by spatial averaging are called dispersion coefficients. In particular, one-dimensional cross-

sectionally averaged models use a longitudinal dispersion coefficient, which is usually found to be related primarily to lateral velocity gradients [Fischer, 1967b, p. 189]. The magnitudes of vertical and lateral diffusion and longitudinal dispersion coefficients vary with location and time in a tidal waterbody.

Models can be programmed to accomodate variable coefficients, but often a constant will be used because it is too difficult, time-consuming, and/or expensive to measure the dispersion at various locations and times in a given waterbody. The development of laboratory and field experiments to quantify dispersion coefficients for steady (river) and oscillating (estuary) flow may be traced through the work of H.B. Fischer, which was reviewed in Fischer [1973].

Many experiments have been conducted in rivers [Nordin and Sabol, 1974] and estuaries [EPA, 1975b, p. 30] in an attempt to relate measured dispersion characteristics to the geometry and flow characteristics of the waterbody. The experiment is usually conducted by placing a tracer, that is, a neutrally buoyant solution of some material such as a dye or a radioactive element, that can be detected in very small concentrations, at a prescribed location and time in the waterbody and then measuring the resulting concentration distribution downstream at one or more subsequent times. These measurements are subject to a great many subjective decisions and experimental variables, which can result in a relatively wide range of experimental values. The investigator usually concludes that the longitudinal dispersion coefficient is primarily a function of the shear velocity and the hydraulic radius, or equivalently a function of the flow velocity, depth, and roughness of the channel.

The use of a two- or three-dimensional numerical model based on tidal-, salinity-, and wind-induced circulation and diffusion, such as the model by Walton (see Chapter 7), for evaluating the flushing time of a complex canal network provides, in addition to a more realistic simulation of water circulation, the advantage of allowing for realistic pollutant inflow distributions. Thus, for example, the effect of an unusually large pollutant point loading of limited duration can be evaluated only by a model that can simulate the circulation and diffusion over successive tidal cycles.

The design of a dye experiment is dictated by the kinds of coefficients used in the model, which in turn depends on the number of spatial dimensions in the model. It has been found by the Hydraulic Laboratory that it is impractical to attempt to measure turbulent diffusion coefficients due to the complexity and variability of the circulation in a canal, and the alternate approach of releasing a tracer and estimating coefficients from the resulting concentration profiles over successive tidal cycles has been adopted. The limitations of this approach are discussed in Section 7.5.2.

2.4.7 Stratification

When the vertical salinity and temperature gradients in a canal are very small, the canal is said to be "well-mixed". However, when these gradients are sufficiently large to affect the circulation in the canal, then the flow is called density- or temperature-induced. When the gradients become so large that two distinct layers of water form, one above the other over a significant length of the canal, the conditions are called "stratified".

A distinction between gradient-induced flow and stratified flow should always be maintained, as they describe two different phenomena. When stratification occurs the two water masses tend to form an interface sloping downward and inward into the canal. In this case the denser water mass is often referred to as a "saline wedge."

A salinity profile is adequate for showing the shape of a density profile provided the water temperature is relatively constant. However, the density of seawater varies with both salinity and temperature, and therefore density should be used when vertical temperature differences of greater than about 5°C are measured at a station.

Density gradients and stratification are indicators of the relative stability of water masses. If the water is well-mixed, the vertical density profiles will be straight vertical lines. If it is completely stratified and in equilibrium, with the less dense water mass wholly above the other more dense layer, the vertical density profile would consist of two vertical profiles of different value, one representing each water mass, and a relatively sharp change from one value to the other at the interface. A sudden decrease in density with depth would indicate that the waterbody was unstable and in the process of overturning. In practice, one usually sees a more gradual change in density with change in depth and it is necessary to examine several vertical density profiles over a period of time to determine whether there is a tendency toward stratification or not.

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 that is located far from the tidal entrance of a canal system, if little circulation is taking place.

Salinity gradients and/or stratification commonly occur in Floridian canals during the wet (summer) season due to runoff from rainstorms. Figure 2.9c from Lindall, Fable, and Collins [1975, pp. 82-83] shows the change in the salinity difference between surface and bottom stations in a Tampa Bay canal system (Figure 2.9a) 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 2.9b) and that DO gradients (Figure 2.9d) 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.

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

If conditions are favorable for stratification in a particular canal, the salt water 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. 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 salt water in the receiving waters. Due to frictional retardation of the movement of the wedge out of the canal, a salt water dome is frequently observed in the canal at low tide. The hydrodynamics of the salt wedge movement will be discussed in Chapter 7.

2.4.8 Geology

The geology of a site controls the movement of surface water, subterranean water, and indirectly the stability of canal channels through the potential for erosion or deposition at a site. In considering a site for development, geologic data are used to obtain

1. the potential water supply. The depth to the aquifer and its potential rate of supply are of importance in determining whether onsite water supply will be sufficient.
3. surface water runoff patterns. Runoff is directed by the topography of the site and the infiltration rate is controlled by the type of ground cover at a particular location, and the geologic formation under the surficial deposits.
4. construction requirements. For construction it is necessary to determine the characteristics of the soil, such as its physical formation, granular content, weight-bearing capacity, and its thickness.
5. canal deposition or erosion rates. The erosion or deposition of canal bank and bottom material is a function of the characteristics of the material, the geometry of the channel, and the water velocity.

2.4.8.1 General Features

Alternating periods of high and low sea level created the landforms of Florida. The coastline is characterized by geologic features which are generally parallel to the coast, which implies that the sea has had much to do with shaping this region [Puri and Vernon, 1964, p. 12]. In fact, by thorium dating it has been determined that the main body of the Florida peninsula was formed about 190,000 years ago [Veri, et al, p. 18] when the peninsula was below sea level and layers of limestone were being formed by the sea. "Today the sea level is rising again at a rate of three inches per 100 years, and land building processes are active along the shore and on the ocean floor" [Parker and Cooke, 1944, in Veri et al, 1975, p. 16].

The generalized landforms of Florida have been divided into the Northern Highlands, Central Highlands, Coastal Lowlands, and the Southern Zone or Distal Lowlands, as shown in Figure 2.10 [Puri and Vernon, 1964, pp. 7-13]. The geologic structures and stratiography of the state are highly variable from one region to another (for example, see Figure 2.11) but for a general view of the primary peninsular bedrock formations and the two principal aquifers, the Biscayne Aquifer and the Floridan Aquifer, see Figure 2.12. As can be surmised from the latter figure, each site will have a unique stratigraphy which will have to be investigated for a particular development.

As far as water resources are concerned, limestone is the most important geologic component of the state. There are 800 miles of limestone extending from Key West to Tallahassee, in layers up to 12,000 ft thick above a granite base [Veri, et al, 1975, p. 18]. Limestone is composed of calcium carbonate which, in Florida, is deposited in various

ways to produce different types such as oolitic limestone and marl. Groundwater erodes these limestone deposits by dissolving the calcium, the rate of erosion being on the order of one foot thickness each 800 to 1,000 years, a fairly rapid rate [Veri, et al, 1975, p. 19].

Five types of limestone are formed in Florida:

1. Oolitic limestone. This type is formed in shallow mud seas by the precipitation of calcium carbonate from the ocean water around a small nucleus, such as a shell, and then cementing together of the small particles.
2. Bryozoan limestone.
3. Coral Reef (Key Largo) limestone.
4. Coquina limestone.
5. Marl. This type is formed by excretion of calcium carbonate by marine organisms in salt water. It is a fine-textured, clay-like deposit that can become hardened over a period of time and, because of a lack of pore spaces, becomes relatively impervious [Veri, et al, 1975, p. 18].

Surficial deposits in Florida consist of organic soils and a variety of types of sand. Pamlico sand is usually highly permeable, Talbot and Penholoway sands less permeable, and the organic soils and marl are the least permeable of all. Pervious surfaces are, of course, the best for aquifer recharge, while the impervious materials limit percolation and result in higher runoff. Since organic soils absorb large quantities of surface water until they become saturated, they generally contribute to runoff only during heavy rainstorms [Veri, et al, p. 19].

Marl is relatively impervious and has poor aquifer potential. Rainfall runs off marl to lower elevations, forming lakes in depressions. The bearing capacity for building varies with thickness of the layer, and there is some potential for shrinking or swelling.

Pamlico sands are highly permeable and are usually found in layers from one to two feet thick. Their water-bearing capacity is low, and they shift easily with the wind and erode easily by water when not revegetated. The physical characteristics and aquifer potential of other South Florida geological formations are summarized by Veri et al [1975, pp. 20 and 24].

2.4.8.2 Bank and Bottom Materials

The magnitude of the bank and bottom roughness, and variations in the roughness with location in the canal network, control in part the dissipation of energy used to overcome friction. The effect of friction on the flow may either be quantified from measured vertical velocity profiles or predicted, for a proposed canal network, from estimates of mean velocity, hydraulic radius, and tidal characteristics at the site.

The beds of the Floridian canals surveyed by the Hydraulic Laboratory consisted either of silt or sand, except in the Florida Keys where broken pieces of limestone deposited by the dredging process are also found. The banks of natural channels have been observed to be made of mostly sand or sandy soil. The stability of these beds and banks is dependent upon the grain size distribution and velocities in the channel. Beds and banks tend to adjust to the quantity of flow in the channel, and the geometries of inlets and channels, i.e., the depths, side slopes, and widths, must be designed for stability under the expected range of flow in a given channel. In addition, the effect of boat traffic on the

stability of channel geometry must be considered during design.

Bed and bank samples were taken in the 57 Acres system and in Frenchman's canal, located about two miles south of Jupiter inlet on the Intracoastal Waterway (ICW). The samples were measured by sieve analysis, and with a hydrometer when a significant portion of the sample passed through the finest sieve available. For each of the samples the grain size distribution was plotted and the value of $d_{35\%}$ was selected as the effective grain size, d_e, where the percentage is the percentage of materials in the sample finer than the given grain-size.

Using the MIT soil descriptions, all bank samples were found to be medium or fine sands, some of which were cemented by organic materials. The bottom sample group consisted of both cohesive and noncohesive samples. The six cohesive samples consisted of medium silts and coarse, medium, and fine clays, while the noncohesive samples ranged from fine to medium sands. The effective grain sizes on the banks ranged from 0.18 to 0.24 mm while the bottom sizes ranged from 0.11 to 0.30 mm (sand) and from .00024 to .0075 mm (silt). There was no significant difference between the samples from the two different locations.

2.5 Water Quality

The water quality in Florida's residential canals may be characterized by a wide variety of constituents. The most significant, from the viewpoint of the canal designer, is dissolved oxygen. The State has established critera for five classes of water, each of which is defined in terms of measured dissolved oxygen (see Section 3.3.2.3). Dissolved oxygen could be simulated with the numerical model (CANNET3D) developed for the canal design project by specifying local sources and sinks and decay rates in the appropriate cells in the network.

2.5.1 Variability of Salinity, Water Temperature, and DO

The literature on the environmental effects of canals and dredged holes contains data on salinity, temperature, and dissolved oxygen which are useful to the canal designer. Some of this literature was first reviewed for a study of the environmental impact of borrow pits in Maryland estuarine water [Polis, 1974]. He summarized reports on work in Texas, Florida, North Carolina, Maryland, Delaware, and New Jersey. Later Bailey [1977] extended this literature survey to 1976, specifically for Floridian canals.

In order to quantify the variability of salinity, water temperature, and dissolved oxygen in Florida's residential canals, several studies with representative data have been chosen from each of the principal coastal areas of the state. Included with these results are published and previously unpublished data collected by the Hydraulic Laboratory in support of this project. It is emphasized that these data are only a sample and far from being a comprehensive selection.

Figure 2.13 shows the approximate locations of the sites at which the compiled data used for the variability summary are located. Each location is shown in detail in the reports referenced in Section 1.4 of Morris, Walton, and Christensen [1978]. Studies made in the same general area, but not in the same canal system, are designated by small letters "a" and "b". It should be realized that canal systems of varying size and age are located in many other coastal communities in Florida besides those indicated in Figure 2.13.

The mean, minimum, and maximum values for each parameter, compiled from a selected set of data, is herein cautiously presented as an indication of the variability of that parameter in Florida. These

values have only limited usefulness and must not be taken as a valid sample, but can be viewed as an indication of the extent of variation of salinity, temperature, and dissolved oxygen in canals in various regions of Florida. While the data are presented separately for the "wet" season (June through September) and the "dry" season (October through May) and for different geological areas, there are many potential discrepancies in the composite data bases. Since five sets of investigators were involved (with collaboration between Bailey and the Hydraulic Laboratory, but essentially separate measurement efforts), there were five different sets of instruments, five different sizes of data bases, and several stages of transcription and analysis for each data base with the involvement of at least five different analysts. Most important of all, however, is the fact that the differences in physical features of the canals, water circulation dynamics, and variations in location and time within the canal system are totally ignored in this collection. The mean, maximum, and minimum values of the three variables are summarized in Table 2.2. The standard deviations of these data are not available since the original data are not all published.

The salinity varies, in the canals represented by this collection of data, from a minimum of 5.3 ppt several miles up the Loxahatchee River (Figure 3.14) to a maximum of 40 ppt in the Frenchman's canal system (Figure 2.15). The former value results from a great deal of mixing with river water. The latter, somewhat higher than the salinity of ocean water, may be caused by high evaporation over a slow moving, poorly mixed portion of the canal waters, or by leaching of minerals from the walls or bottom of the canals. Other explanations are proposed

by Griffin [in Morris, et al, 1977b, Appendix A, pp. 7-10]. The high values of salinity could also be attributed to measurement error. In any case, as will be discussed in more detail later, the more interesting consideration with regard to salinity is whether or not it varies with depth in different locations in the canal system. Since density is directly proportional to salinity, zones of different salinity will, if not arranged in stable layers, induce density currents which tend to stabilize the waterbody. Since density currents will advect pollutants in the water, just as will any other currents, these should be taken into consideration in a predictive model when significant in the prototype. If a waterbody has no variations in density it is said to be homogeneous or well mixed, a simplification frequently, and sometimes unjustifiably, introduced in the development of numerical models.

Water temperature in the canals included in Table 2.2 varies between 21 and 37°C. The winter or "dry" season temperature band lies completely below the summer, or "wet" season band, as would be expected. As is the case with salinity gradients, vertical gradients in temperature can also induce density currents which must be included in a model if they are found to be significant in the prototype. Due to the decrease in density with increasing temperature (a rate of 34×10^{-5} ppt per 1°C at 30°C), the water column tends toward stability when heated from above, and toward instability when the surface cools suddenly.

Dissolved oxygen (DO) in the canals in Table 2.2 varies from 0 to 16.3 ppm. It is generally accepted that a minimum of 4 ppm (Section 17-3, Florida Administrative Code) or 5 ppm (in the opinion of some biologists) is necessary to support fish and most other aquatic

creatures. It has frequently been observed in both wetland and upland canals, that anaerobic (approaching zero ppm DO) conditions can develop, particularly at the bottoms of deep canals or holes. These conditions develop whenever the supply of oxygen from the surface of the water is not sufficient to replenish that which is consumed in the water column, either by the respiration of aquatic life or by decomposition of organic materials. If sufficient oxygen is not provided at the bottom, anaerobic decomposition of organics will occur accompanied by the release of hydrogen sulfide gas. At night, oxygen in the water is depleted by respiration, while during the day it is restored by photosynthesis. Thus, the DO critieria set by the state are expressed as not less than an average value of 5 ppm over a period of twenty-four hours.

The saturation concentration of dissolved oxygen in water decreases with increasing temperature and salinity from about 14.6 ppm in fresh-water at 0°C to about 6.1 ppm in 36 ppt seawater at 30°C. The rate at which oxygen is taken up by water at the surface, the reaeration rate, is proportional to the difference between the saturation value of DO and the actual value near the surface. The reaeration rate increases as the turbulence of the water increases, due to entrainment. Sinks of DO include respiration and, more importantly, the oxygen demand of benthic materials. This will be on the order of 0.05 g-O_2/m^2/day [Isaac (1965) and Servizi, et al (1969), in Polis, 1974, p. 44] depending on the type of material being oxidized, and increases with temperature. A transient sink of considerably greater magnitude occurs upon a sudden resuspension of bottom sediments, which can be caused by dredging or by a storm. It is evident that anaerobic conditions occur naturally at some locations

in natural channels, but the state regulations for dissolved oxygen in canal waters permit lower limits only if their prior existence can be proven.

Low DO concentrations have been associated with fish kills in Floridian canals. It is thought that fish kills caused by low DO can result only when fish become trapped in a canal or cannot find their way out of a confined area, such as in a dead-end canal, since it is known that they will tend to abandon an area of deteriorating DO long before lethal conditions prevail.

Bailey [1977] has attempted to find statistical relationships between water quality parameters and the physical characteristics of the forty-six canals included in his data set. For the parameters listed in Table 2.3 these results may be summarized as follows:

1. Regression equations for average and minimum dissolved oxygen concentrations explained ninety-one and eighty-eight percent, respectively, of the observed variabilities
[Bailey, 1977, p. xiii].

2. A simple water quality index would not be adequate to classify these canals, since the first principal component explains just twenty-nine percent of the total variability of these water quality parameters. [Furthermore,] the first three components or factors can account for [only] sixty-three percent of the differences in water quality...
[Bailey, 1977, p. 107-110].

The results in the second statement above come from a principal component analysis, a statistical technique that identifies the parameters that vary the most throughout a data set and quantifies how much of the variability in the data can be accounted for by successive linear combinations of the variables.

Bailey also used a canonical (linear combinations) correlation

analysis, that indicates the amount of correlation or association between a series of linear combinations of data from two data subsets. Bailey's canonical analysis indicated a substantial association between water quality parameters, and the physical characteristics. Specifically, it was found that,

> 3. a. large [surface area] canal systems tend to have higher average dissolved oxygen concentrations
>
> b. increases in canal width increase average oxygen levels; but [increases in] the product of canal depth and cummulated tidal amplitude tends to reduce the oxygen level
> [Bailey, 1977, P. 168].

Bailey's results are covered in some detail here because they appear to be the most comprehensive statistical analysis of canal data yet attempted, and because the analysis illustrates a very important concept. The results show that, while interesting and plausible relationships between variables and combinations of variables do exist in the data, there is still missing an explanation of the fundamental physical processes and cause-and-effect relationships at work in even a simple, straight, dead-end canal. Thus, when it becomes necessary to explain these results on an objective basis, one can only resort to conjecture. The method provides no guidance for canal design except an analysis procedure which will permit the canal designer to predict the results of his design on a statistical basis, under the assumption that the effects of environmental variables that are omitted either from the designer's model or from Bailey's model are the same, and that all variables are within the ranges of data in Bailey's model.

2.5.2 Sources of Pollution in Canal Systems

In recent years the term "pollution" has often been used, and

misused, for almost any observed constituent in both natural and artifical waterbodies, whether or not that constituent occurs there naturally. Pollution may be defined in a general way as the degradation of water quality with resultant significant interference with beneficial water use [Haney, 1966]. Thus, for canal work a pollutant might be defined as any material that would not ordinarily be found in the "receiving" or communicating waterbody and which is considered to be present in sufficient concentration to constitute a hazard to aquatic or human life. Pollution is also defined, in an indirect way, by state regulations regarding water types (see Section 3.3.2).

2.5.2.1 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 meteorological 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 2.3 summarizes the principal pollution sources and the types of constituents that may be found in residential canal systems. Note that in the category of meteorological 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 interreactions 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 2.4 summarizes the harmful effects of pollutants in canal waters.

The purpose of the canal model (CANNET3D) is to provide a hydrodynamic framework for simulating the movement and resulting concentrations of a single pollutant constituent. The constituent can be introduced into or taken out of any cell in the model at either a constant rate, or at a predetermined variable rate. Furthermore, this constituent can be provided with any rate of decay in any cell (although this decay coefficient cannot be varied with time, only spatially). Thus, if rates of change and inflow and outflow rates can be specified for the lateral inflow at any cell, the model can be used to simulate the effects of changes in concentration that occur as a result of other than hydrodynamic forcing functions.

2.5.2.2 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. An example using population density in terms of type of dwellings and lot size is given in Table 2.5, and a set of average residential water use and sewage production data are shown in Table 2.6. Here the water usage rate increases from less than 125 to 167 gal/capita-day (gal/c-d). Steller, citing figures from Nicholas and Blowers [1974, in Steller, 1976, p. 58] gives an average of 145 gal/c-d for South Florida,

and rates varying from 150 to 761 gal/c-d for different parts of Collier County, Florida [from Veri, 1972, in Steller, 1976, p. 60]. The differences in water consumption are accounted for, according to Steller [1976, p. 58], in terms of the age of the development and the development type. Water use evidently tends to decrease with increasing age of development [Miller, 1975 in Steller, 1976, p. 58], and those types of development that encourage landscaped lots and swimming pools have increased water consumption, although not necessarily a concomitant increase in sewage water. Lower rates of domestic water consumption, 15 to 70 gal/c-d, presumably averaged for the entire country (but also a decade older) are cited by Fair, et al [1966, p. 5-13].

The volume of domestic sewage is given as 80 percent of water consumption in Table 2.6. Goodman and Foster [1969, in Eckenfelder, 1970, p. 60] give lower total volumes of sewage production than in Table 2.6, varying from 70 to 100 gal/c-d depending on the type of housing (Table 2.7). In this latter table the volume of five-day BOD (biochemical oxygen demand) is given two values, 90.7 and 77.2 gram/c-d. Fair, et al [1966, pp. 3-9] state that "about 70 percent of the water brought into a community must be removed as spent water. The average flow in sanitary sewers is about 100 gal/c-d in North America."

One set of measurements of the characteristics of municipal sewage is shown in Table 2.8. In this data set BOD accounts for 10 percent of the total mean concentrations. The variability of domestic sewage for "areas of moderate size" is given in Table 2.9.

2.5.2.3 Septic Tanks

The principal factor governing the effectiveness of septic tank disposal systems is, in most instances, the type of the soil in the

vicinity of the absorption field [EPA, 1975b, p. 173]. Soil permeability, soil depth, ground water level, the slope of the surface of the ground, the proximity to surface waters and the presence of fractures or caverns in the geological strata all contribute to the efficiency of the system in filtering waste water into the aquifer and/or into adjacent canals. EPA states that any regulation on septic tank installation should be based on: (1) soil type, (2) horizontal distance from adjacent waterbodies and, (3) vertical distance from the surface of the ground water [EPA, 1975b, p. 174]. It is further recommended that septic tank/sorption fields should be "no closer than 100 ft from a surface waterbody and that these fields be 3 to 4 ft above the saturated soil zone at the wettest period of year" [EPA, 1975b, pp. 174-175].

It is generally agreed that, under ideal soil and hydrological conditions, bacteria and viruses can be effectively removed by "percolation through several feet of fine, unsaturated soil" [EPA, 1975b, p. 177]. The type of soil "not only affects the rate of travel [of these contaminants] but also the degree of reduction of bacterial contamination with distance" [EPA, 1975b, p. 177]. Reports of distances traveled by coliform bacteria range from 10 to 2,000 ft [EPA, 1975b, p. 177].

No general conclusions can be drawn as to the "normal" pollution loading from septic tanks into canals, as there are too many variables to permit a simple answer. In the ideal case, all waste water will infiltrate into the groundwater and the filtering provided by the soil could remove most of the polluting substances. If, on the other hand, a septic tank were not installed in the proper kind of soil, and became

clogged and overflowed, then it is conceivable that most of the waste water entering the septic tank would be introduced into the canal network through overland flow. Therefore, it could be postulated that the range of discharge of pollutant point sources associated with residences on canals could vary between zero and

$$q_I = 90 \frac{\text{gal}}{\text{cap-day}} \times 3 \frac{\text{cap}}{\text{lot}} \times \frac{1}{7.481} \frac{\text{ft}^3}{\text{gal}} \tag{2.10}$$

$$= 36 \text{ ft}^3/\text{lot-day}$$

$$= 40 \text{ ft}^3/100 \text{ ft lot-day}$$

$$= 0.4 \text{ ft}^3/\text{ft canal length-day}$$

assuming 100 ft canal frontage. If a two-dimensional (horizontally averaged) model is used and there are 100 ft lots on both sides of the canal, the maximum inflow of waste water (no septic tanks operating) would be on the order of

$$q_I = 2 \times 0.4 \frac{\text{ft}^3}{\text{ft-day}} \times \frac{1}{24} \frac{\text{day}}{\text{hr}}$$

$$= 0.03 \frac{\text{ft}^3}{\text{ft-hr}}$$

these values were used in most simulations of pollutant inflow.

The concentration of a typical residential BOD coefficient is estimated to be on the order of 90.7 g/cap-day (Table 2.7) or

$$c_I = \frac{90.7 \text{ g/cap-day}}{90 \frac{\text{gal}}{\text{cap-day}} \times 3.785 \frac{\text{liter}}{\text{gal}}} \tag{2.11}$$

$$= 0.267 \text{ g/liter or } 267 \text{ ppm}$$

These values were used in most simulations of pollutant inflow.

2.5.2.4 Boats and Marinas

The effects of operation of outboard engines on the quality of water and the life systems in a canal would be negligble 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 movement 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 attention to water movement and 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.

2.6 Ecosystem Components

As this study is concerned principally with the hydrodynamics and transport mechanisms of Floridian canal systems, the effects of the canals on the internal functioning of various ecosystems is beyond the scope of the project. However, those interactions which must be defined

quantitively in the process of developing a canal design will be described here.

2.6.1 Algae and Plants

Marine algae, grasses, shrubs and trees are vital in many ways to canals. Primarily, they

1. provide a sink for excess nutrients in the water,
2. provide a food source for herbivorous creatures,
3. offer a hiding place for the juveniles of many species and some smaller animals,
4. secure bottom sediments and side slopes against high velocity currents and storms,
5. provide photosynthesis during sunlight hours.

Thus, a canal with a well-designed distribution of plant life will be less turbid, biologically more productive, more stable, and will act as a better nutrient filter than one with little plant life or with a random selection of plant life.

Many species of vascular plants and algae are associated with Floridian canals. In the Florida Keys, for example, Chesher [1974, p. 97] identified 104 species of plants in natural and man-made canals. Investigations on the productivity and nutrient uptake of certain abundant and obviously important species, such as mangrove, turtle grass, and marsh grass, have been undertaken by other researchers and can be utilized qualitatively in rational canal design.

After a canal has been dredged, whether during construction of the system or for maintenance purposes, it will take one or two years for re-establishment of vegetation. It is therefore necessary to introduce the desired species of plants into the locations which require

stabilization, particularly the canal banks, as soon as possible after dredging. There is an extensive literature on transplanting and establishing the important types of vegetation listed above [Snyder, 1976, pp. F1-F5].

The depths at which plants can grow in canals vary with the turbidity of the water, the amount of hydrogen sulfide which leaches through the aquifer into the canal, and its flushing characteristics [Chesher, 1974, p. 109]. Chesher has found algae at a depth of 12 ft in one canal, and only small amounts of algae at 7 ft in another canal. It was also shown that, in the Florida Keys at least, every species of plant except one that was found in natural canals was found also in man-made canals.

2.6.2 Lawn Grasses

Lawn grasses lower the rate of flow of stormwater and runoff, thus increasing the rate of uptake of nutrients. Since the direction of flow can be controlled by filter mounds and swales (Figure 2.16), and the rate of flow by the slopes of the swales, the nutrient uptake capability of lawn grass should be considered in design of drainage for a particular design storm [Carlson, 1974, in Snyder, 1976, pp. 61-62].

2.6.3 Mangroves

Mangroves are a dominant natural feature of the south Florida coastline, one species of which, the black mangrove, extends as far north as the Crescent Beach area on the east coast. The mangrove, unlike some other species, has clearly defined roles in southern Florida ecosystems, which only during the past decade were beginning to be widely appreciated. The natural roles of mangrove ecosystems, as summarized by Lugo, A.E., et al [1971, in EPA, 1973, pp. XIX-12],

include the five general functions listed in Section 2.6.1 plus:

1. They have a natural tendency to propagate into nearby waters and, in the process of trapping sediments and putting new roots down, to build new islands. However, their island-building capability is limited by their susceptibility to erosion around the roots. Thus, boat wakes can cause erosion of mangrove during revegetation work, and rip-rap, filter cloth, and flow and/or wave deflectors are needed to eliminate this problem.
2. They provide oxygen to their root systems to maintain root respiration and oxygenate the low-oxygen mangrove substrate.

The "classical" mangrove ecosystems are composed of three dominant species. Proceeding from deeper water to the shore, the Red Mangrove *(Rhizophora mangle)* is found with its prop roots almost always covered by the tide, in water of highest salinity. Shoreward of the Red Mangrove is the Black Mangrove *(Avicennia germinans)* growing in shallower depths and subjected to smaller diurnal tidal fluctuations, followed by the White Mangrove *(Laguncularia racemosa)* and Buttonwood *(Conocarpus erectus)* which are influenced only by extreme high and storm tides. While these species are usually found in close association, they can also be found separately.

Transplantation of mangrove has been satisfactorily accomplished by canal property owners, as observed at the Cudjoe Gardens canal system in the Florida Keys by Hydraulic Laboratory personnel.

2.6.4 Turtle grass

The primary contribution of turtle grass *(Thallasia testudinum)* is the stabilization of bottom sediments by its extensive root system. The blades of grass act as sediment traps, and this in combination with the uptake of nutrients which would otherwise be used by plankton, reduces turbidity in the canals [Chesher, 1974, p. 108]. After a canal has first been dredged there will be, for a year or two, no natural vegetation on the bottom. In the Florida Keys cuban shoal weed *(Diplanthera wrightii* is the first colonizer after dredging, becoming established in about two years. Following *Diplanthera, Thallasia* requires an additional two or three years to develop an intitial population [Chesher, 1974, p. 108]. It is therefore necessary to plan the mix and distribution of intertidal, bank and bottom vegetation, and to transplant these species to the site as soon as possible after the channels have been completed.

The production, seasonal variations, and salinity tolerance of *Thallasia* are reviewed by Zieman [1975; in Cronin, 1975, pp. 541-562]. He finds an optimal salinity of about 30 ppt and an optimal temperature of 30°C. Its salinity tolerance range is from 3.5 to 5 ppt up to as high as 60 ppt for short periods of time, and its temperature range is about 20 to 35°C [Zieman, 1975, in Cronin, 1975, p. 553]. *Thalassia* stocks fluctuate with the season, but normally do not completely die out in winter.

2.6.5 Marsh Grass

Natural tidal marshes have become recognized as a valuable resource. They serve not only as a nursery for a large number of sports and commercial fishery species, but additionally in stabilizing shorelines and affording protection from storms to developed areas by absorbing and

dissipating wave energy and temporarily storing water. Although dredging for development and navigation has destroyed substantial areas of marsh in Florida, much of the marsh that has developed in recent years has grown on dredge spoil deposits in the vicinities of estuaries. Efforts to restore or create new marshes on a variety of substrates has been demonstrated to be practical and effective [Woodhouse, W.W., Jr., et al, 1974, p. 11], although these grasses also need protection from erosion during initial establishment.

Spartina alterniflora (smooth cordgrass) is the dominant flowering plant found in regularly flooded intertidal marshes along the Atlantic and Gulf Coasts. It can grow in a wide range of substrates from coarse sands to silty-clay sediments. It appears to be well adapted to anaerobic soils, but does not usually reach its maximum growth in higher salinity (35 ppt) waters. It is found, often with a related species (giant cordgrass *S. cynosuroides*), in freshwater tidal marshes.

S. alterniflora propagates into new areas by vegetative means as well as by seeds. Pieces of marsh, when dislodged, may float to a new, bare location, take root, and spread. Transplanting is effective with seedlings or with natural stands, but handling and germination from seeds appears to be uneconomical [Woodhouse, W.W., Jr., et al, 1974, p. 19].

Table 2.1 - Chances of Hurricane Force Winds in any Given Year.

City	Chances
Jacksonville	1 in 100
Daytona Beach	1 in 50
Melbourne - Vero Beach	1 in 20
Palm Beach	1 in 7
Miami	1 in 6
Key West	1 in 8
Fort Myers	1 in 11
Tampa - St. Petersburg	1 in 25
Apalachicola - St Marks	1 in 17
Pensacola	1 in 8

Source: Bradley, 1974, p. 51.

Table 2.2 - Mean, Maximum, and Minimum Values of Salinity, Water Temperature, and Dissolved Oxygen in Floridian Canals.

COASTAL AREA	LOCATION	REF	NAME CODE	LOC CODE	JUNE THROUGH SEPTEMBER									OCTOBER THROUGH MAY								
					SALINITY, ppt			WATER TEMPERATURE, °C			DISSOLVED OXYGEN (ppm)			SALINITY, ppt			WATER TEMPERATURE, °C			DISSOLVED OXYGEN (ppm)		
					AVE	MIN	MAX	AVE	MIN	MAX	AVE	MIN	MAX	AVE	MIN	MAX	AVE	MIN	MAX	AVE	MIN	MAX
Panhandle	Panama City	B	PA	1a	15.3			29			7.4	6.4	9.1									
		E	PA	1b	24.5	23.1	27.8	28	28	29												
Central West Coast	Boca Ciega Bay	B	BC	2										16.4			25			7.1	3.1	9.4
	Apollo Beach	B	AB	3	14.8			30			7.0	3.7	8.9									
	Venus Waterway	H	VW	4										14.1	12.7	16.2				5.3	3.5	7.4
	Port Charlotte	B	PC	5				28			5.8	0.1	16.0				21			5.8	0.0	10.6
	Punta Gorda	B	PG	6				28			5.1	1.8	12.0				23			7.5	1.3	11.8
		E			35.8	35.2	36.8	30	29	32	2.9	0.0	8.7	27.7	26.0	29.0	22	21	24	5.5	4.5	6.3
South West Coast	Marco Island	B	MI	7										22.5			24			6.0	4.8	7.5
		E						31			4.8	0.0	9.8									
		M			31.8	29.1	33.9	29	27	32	4.8	2.3	6.1	35.2	35.0	35.5	23	22	26			
Florida Keys	Big Pine Key	E	BP	8	35.0	33.7	35.6	32	31	35	5.1	1.1	8.0	37.8	37.0	38.0	24	24	25	5.3	3.0	6.4
	Cudjoe Gardens	H	CG	9																4.7	3.5	5.9
	Marathon	E	MA	10a	31.0	29.7	31.9	31	30	32	3.6	0.6	7.4									
		B	MA	10b	27.0			28			5.5	3.5	6.1									
South East Coast	North Miami	B	NM	11										15.3			27			2.2	0.0	9.8
	Pompano Beach	B	PB	12				31			5.7	0.7	16.3				23			5.1	1.9	8.8
	Hillsboro Inlet	B	HI	13										20.1			26			7.3	5.4	12.8
Central East Coast	Frenchman's Canal	H	FC	14										30.2	19.0	40.0	26	23	27			
	57 Acres	H	57	15							4.3	1.6	6.0							4.8	0.0	10.6
		S				21.7	30.2					1.5	6.0		19.4	31.8					0.0	10.4
	Loxahatchee River	B	LR	16				30			4.6	0.9	7.4				23			5.7	0.9	7.5
		H			12.6	5.3	20.7	32	30	37												
North East Coast	Flagler Beach	B	FB	17	20.5			27			5.9	0.0	11.1									
		MIN/MAX SUMMARY				5.3	36.8		28	37		0.0	16.3		12.7	40.0		21	27		0.0	12.8

References for data keyed to letters in REF column:

B: Bailey, W. A., 1977, pp. 16-38 and 93-97.

E: Environmental Protection Agency, May 1975, pp. 16-25, 67-72, & 82-85.

H: 1. Morris, F. W., et al, August 1975, pp. 39.
2. Christensen, B.A. and Slack-Smith, G., August 1975, pp. 4,6, & 34.
3. Walton, R., et al, October 1975, p. 15 & 16.
4. Walton, R., et al, December 1975, p. 29 & 31
5. Morris, F. W., (this report) Appendix A.

M: McKeehan, D. S., 1975, pp. 64-89.

S: Snyder, R. M., May 1976, pp. 55 & 59.

Table 2.3 - Summary of Possible Sources of Pollution to Canal Surface Waters.

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 indostrial 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

Source: McGauhey, 1968, p. 86.

Table 2.4 - 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 vegetation, lead to eutrophication of water.

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

Table 2.5 - Common Population Densities, Residential Areas.

Type of Dwelling	Persons/acre
Single family dwellings, large lots	5-15
Single family dwellings, small lots	15-35
Multiple family dwellings, small lots	35-100

Source: Fair, Volume I, 1966, p. 5-12.

Table 2.6 - 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 2.7 - 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.

Table 2.8 - Average Characteristics of Municipal Sewage.

	Max	Mean	Min
pH	7.5	7.2	6.8
Settleable solids (mg/l)	6.1	3.3	1.8
Total solids (mg/l)	640	453	322
Volatile total solids (mg/l)	388	217	118
Suspended solids (mg/l)	258	145	83
Volatile suspended solids (mg/l)	208	120	62
Chemical Oxygen demand (mg/l)	436	288	159
Biochemical Oxygen demand (mg/l)	276	147	75
Chlorides (mg/l)	45	35	25
TOTAL (mg/l)		1408.3	

Source: Hunter and Henkelekian, 1965, in Eckenfelder, W.W., Jr., 1970, p. 58.

Table 2.9 - Relationships for Variability of Domestic Sewage from Areas of Moderate Size.

	Multiplier on Quantity in Column		
Type of Flow	Average Daily Flow	Maximum Daily Flow	Minimum Daily Flow
Maximum Daily	2X		
Maximum Hourly	3X	1.5X	
Minimum Daily	2/3X		
Minimum Hourly	1/3X		1/2X

Source: Fair, et al, Volume I, 1966, p. 5-24.

Figure 2.1 - Florida's Coastal Zone as Defined by the Florida Coastal Coordinating Council in 1971.

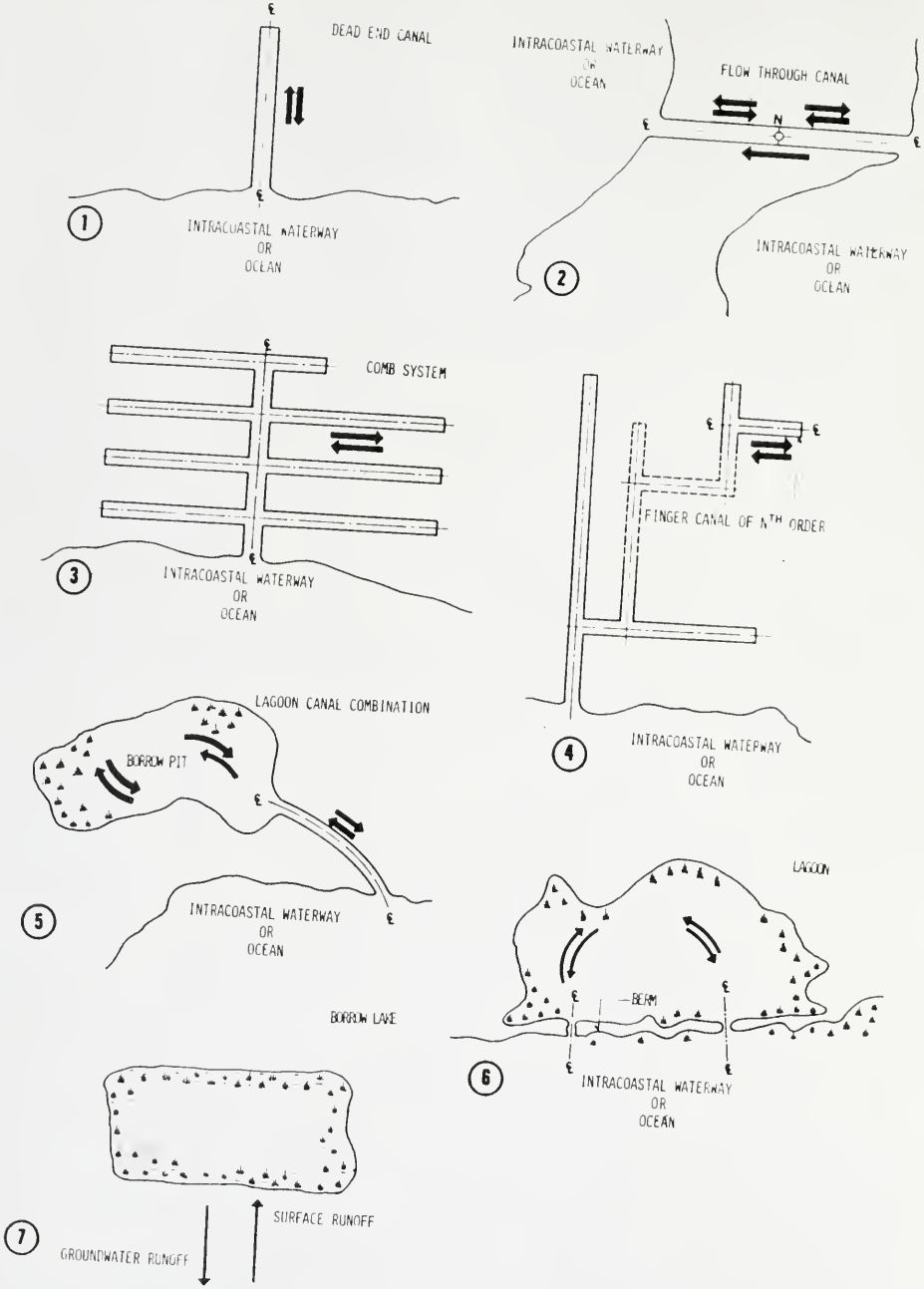


Figure 2.2 - A Geometric Classification for Types of Canals.

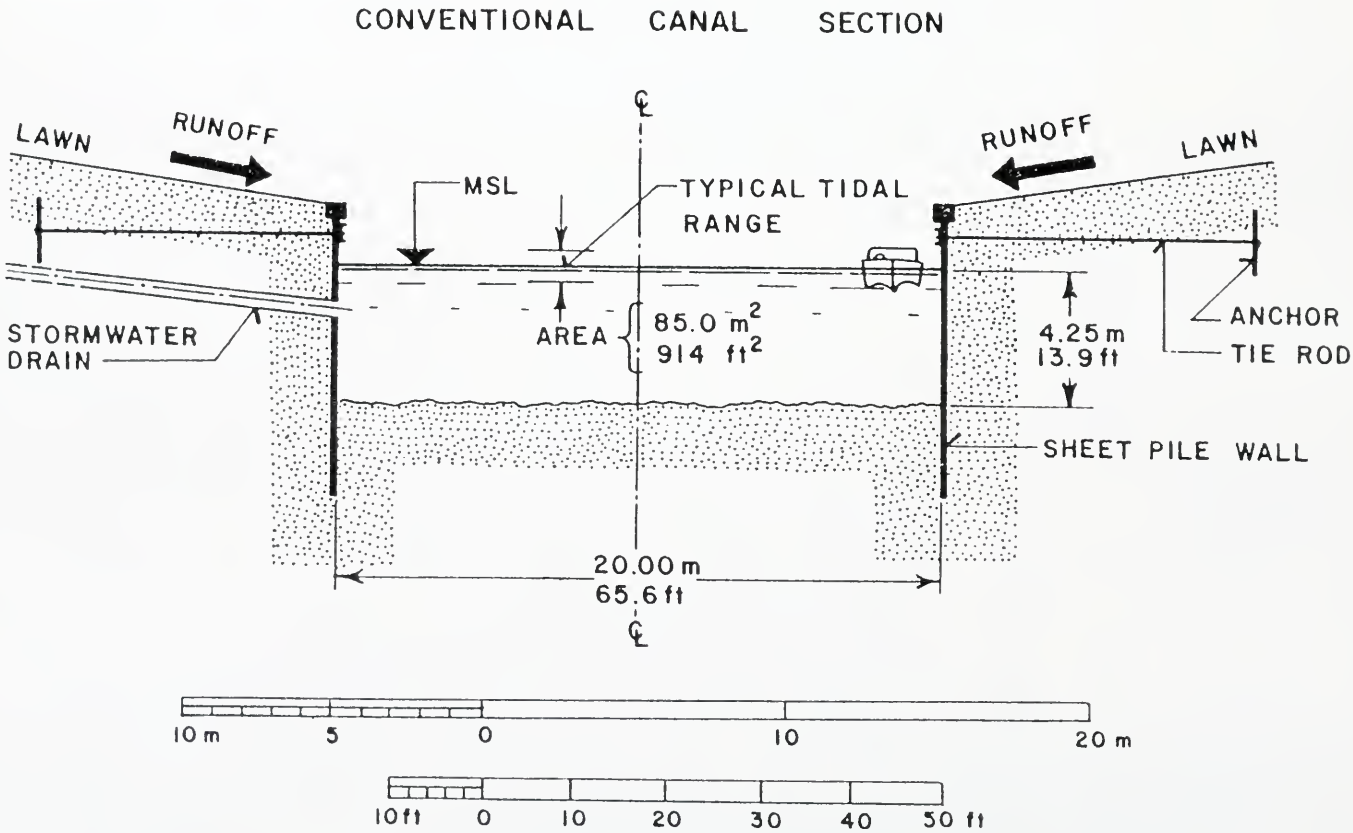


Figure 2.3 - Typical Cross-Section of Conventional Residential Canal (Source: Christensen and Snyder, 1978, p. 3).

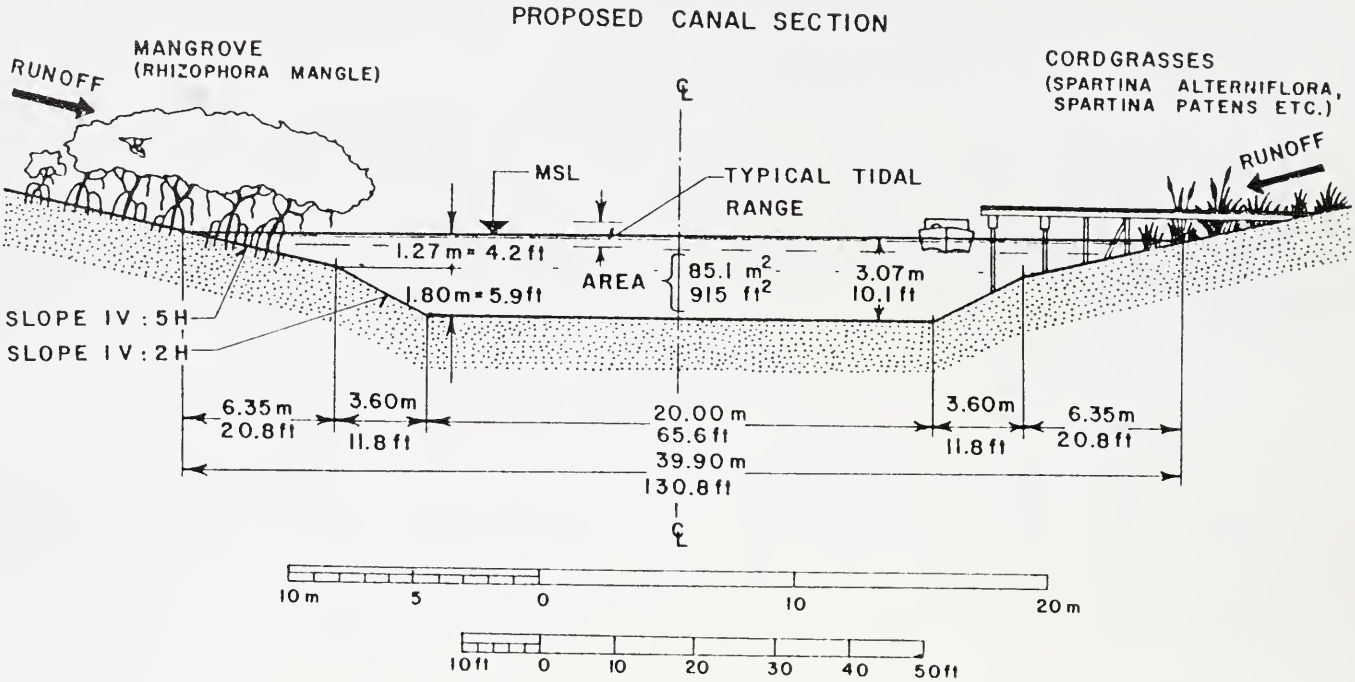


Figure 2.4 - Proposed Canal Section with Same Area as the Conventional Canal Section Shown in Figure 2.3 (Source: Christensen and Snyder, 1978, p. 5).

Figure 2.5 - Classification of Florida Tides.
(Source: Piccolo, 1976, p. 8).

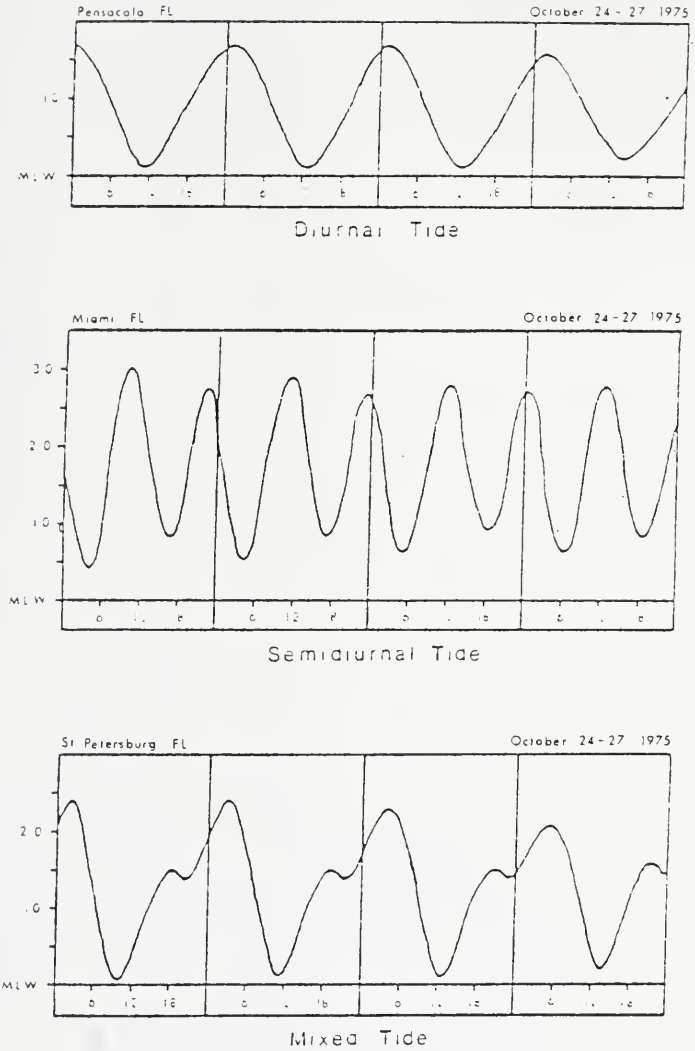


Figure 2.6 - Types of Tides (Source: Piccolo, 1976, p. 4).

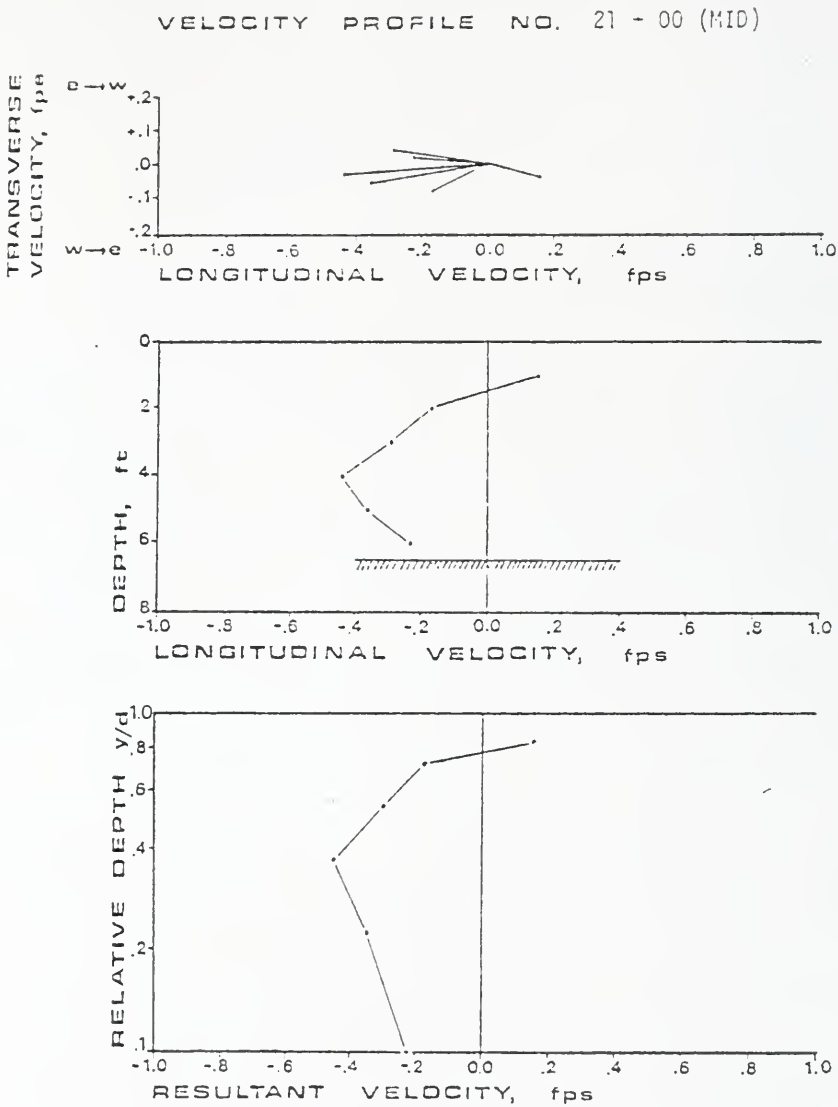


Figure 2.7 - Velocity Profile Measured During Loxahatchee North Canal Field Survey, June 1977.

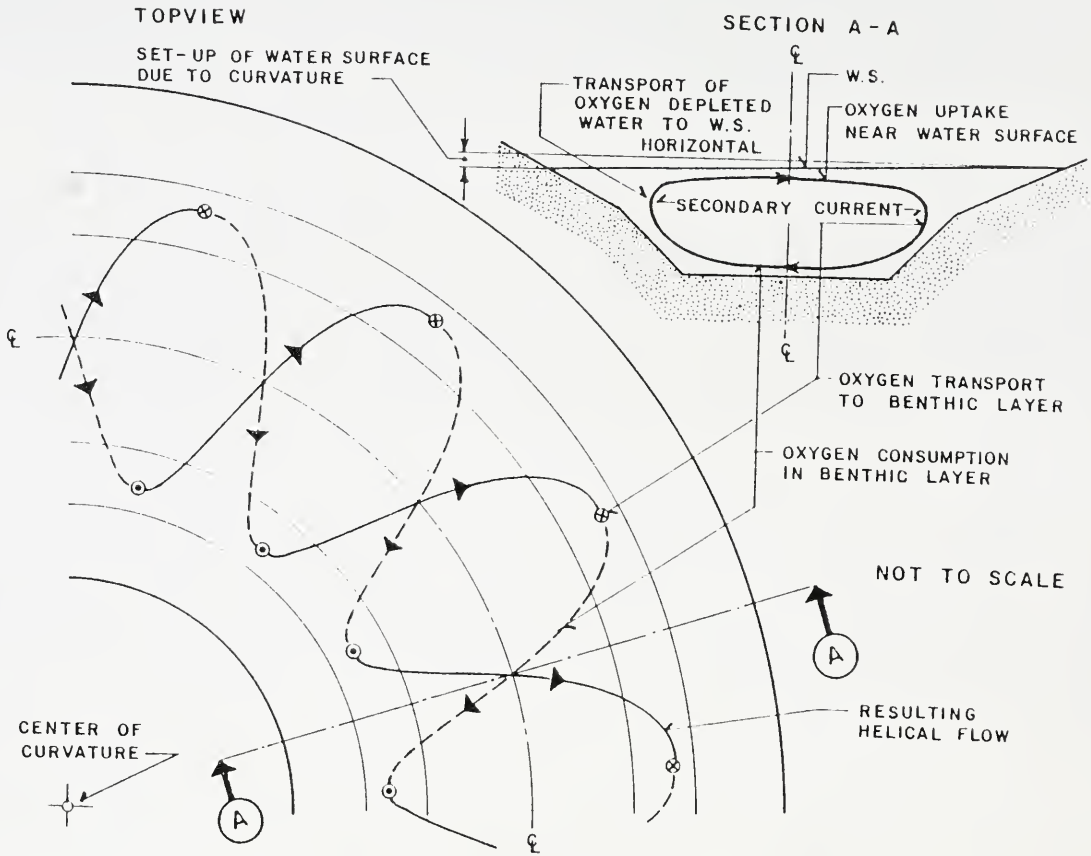


Figure 2.8 - Secondary Current and Resulting Helical Flow in Canal Bend (Source: Christensen and Snyder, 1978, p. 6).

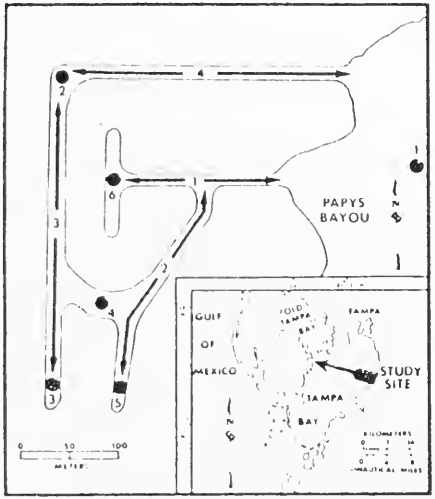


Figure 2.9a - Tampa Bay, Florida, Showing Location of Study Area. Hydrologic Sampling Stations are Shown by Arrows (Source: Lindall, Fable, and Collins, 1975, p. 82).

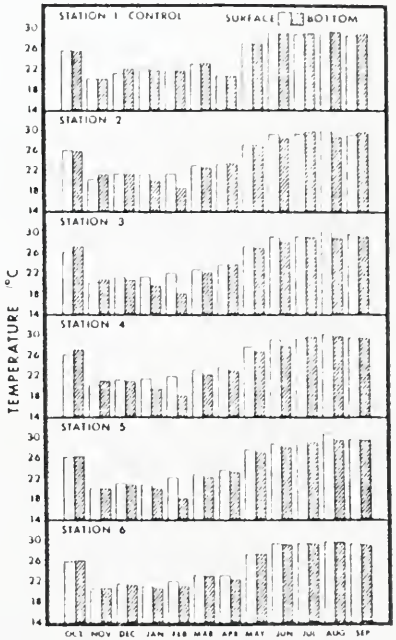


Figure 2.9b - Monthly Water Temperature at the Surface and Bottom of all Hydrologic Stations, October 1971 - September 1972 (Source: Lindall, Fable, and Collins, 1975, p. 82).

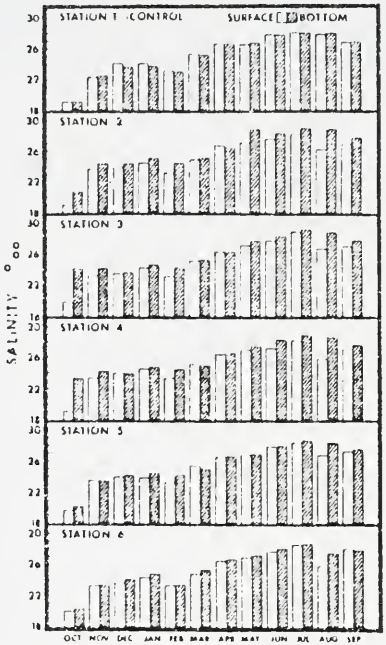


Figure 2.9c - Monthly Salinity at the Surface and Bottom of all Hydrologic Stations, October, 1971 - September, 1972 (Source: Lindall, Fable, and Collins, 1975, p. 83).

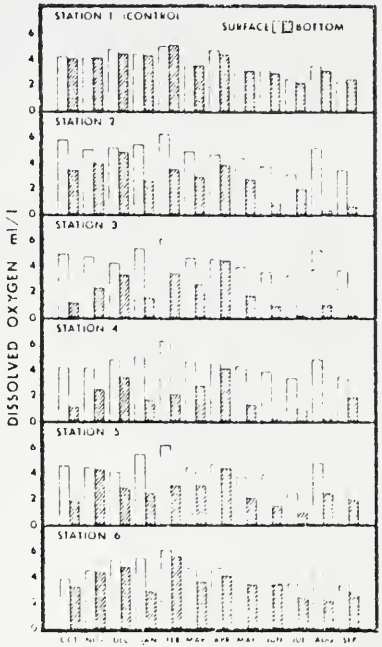


Figure 2.9d - Monthly Dissolved Oxygen at the Surface and Bottom of all Hydrologic Stations, October, 1971 - September, 1972 (Source: Lindall, Fable, and Collins, 1975, p. 83).

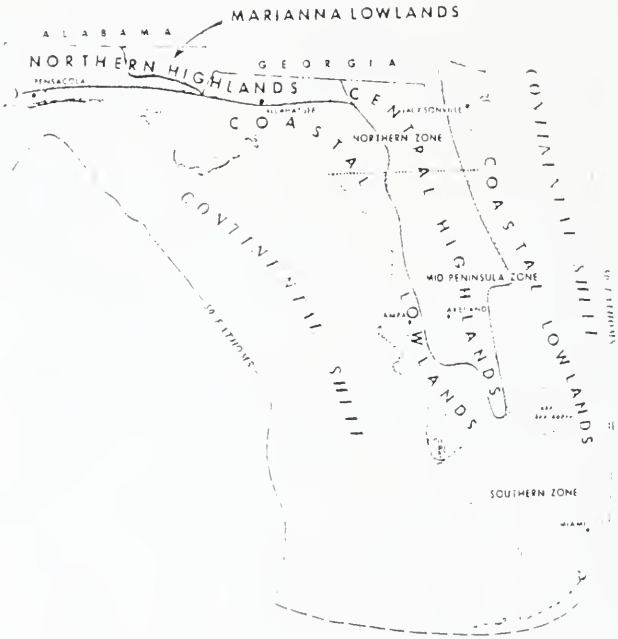


Figure 2.10 - Generalized Locations of Landforms in Florida.
(Source: Puri and Vernon, 1964, p. 3).

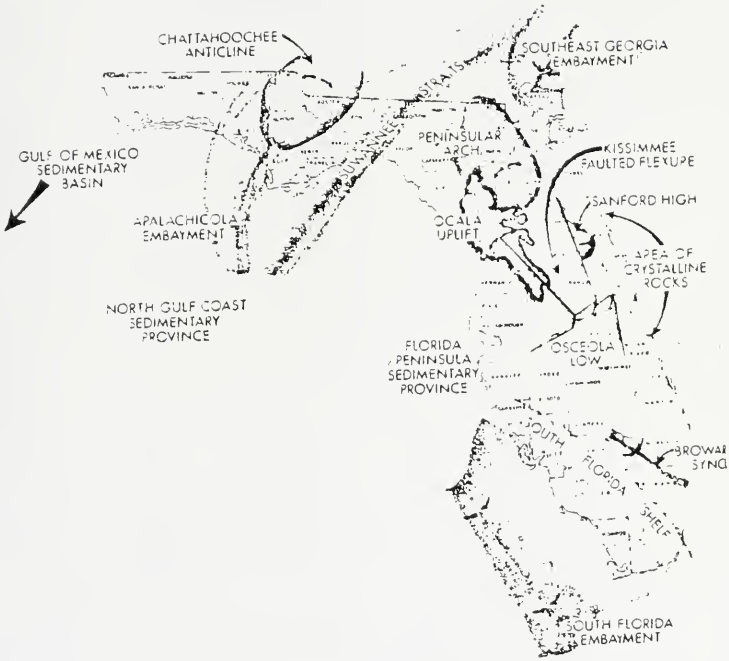


Figure 2.11 - Index to Principal Geologic Structures in Florida. (Source: Puri and Vernon, 1964, p. 4).

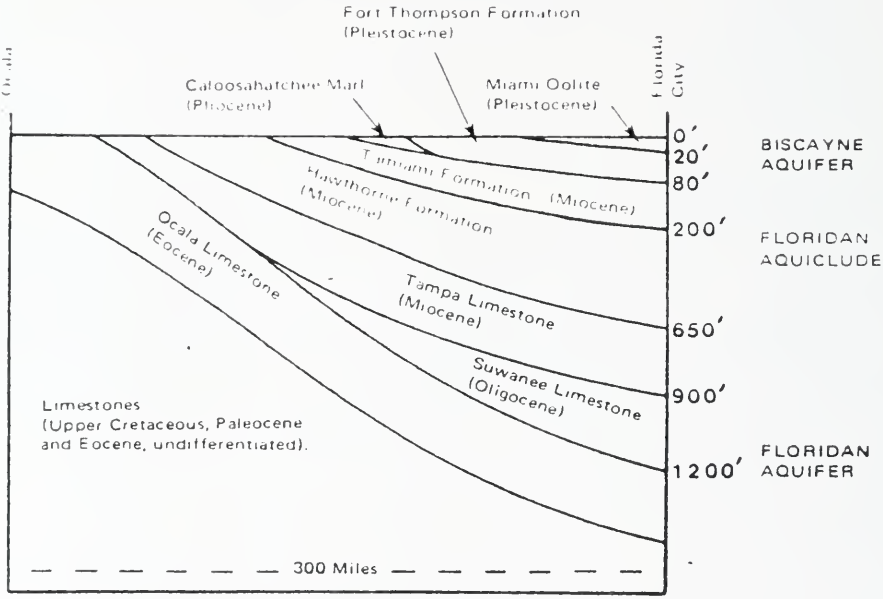


Figure 2.12 - General Layering of the Bedrock Formations Below Southern Florida from Ocala (North) to Florida City (South) (Source: Parker, et al, 1955, p. 63 in Veri, et al, 1975, p. 25).

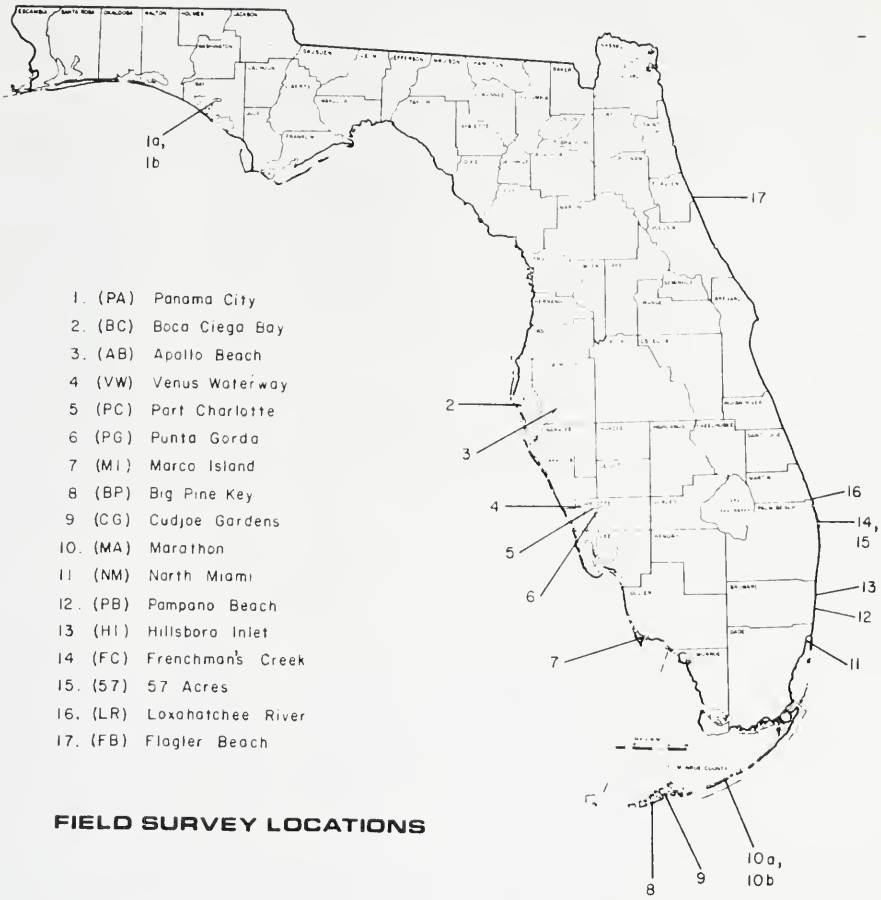


Figure 2.13 - Location Map for Sources of Data Used by Bailey (1976).

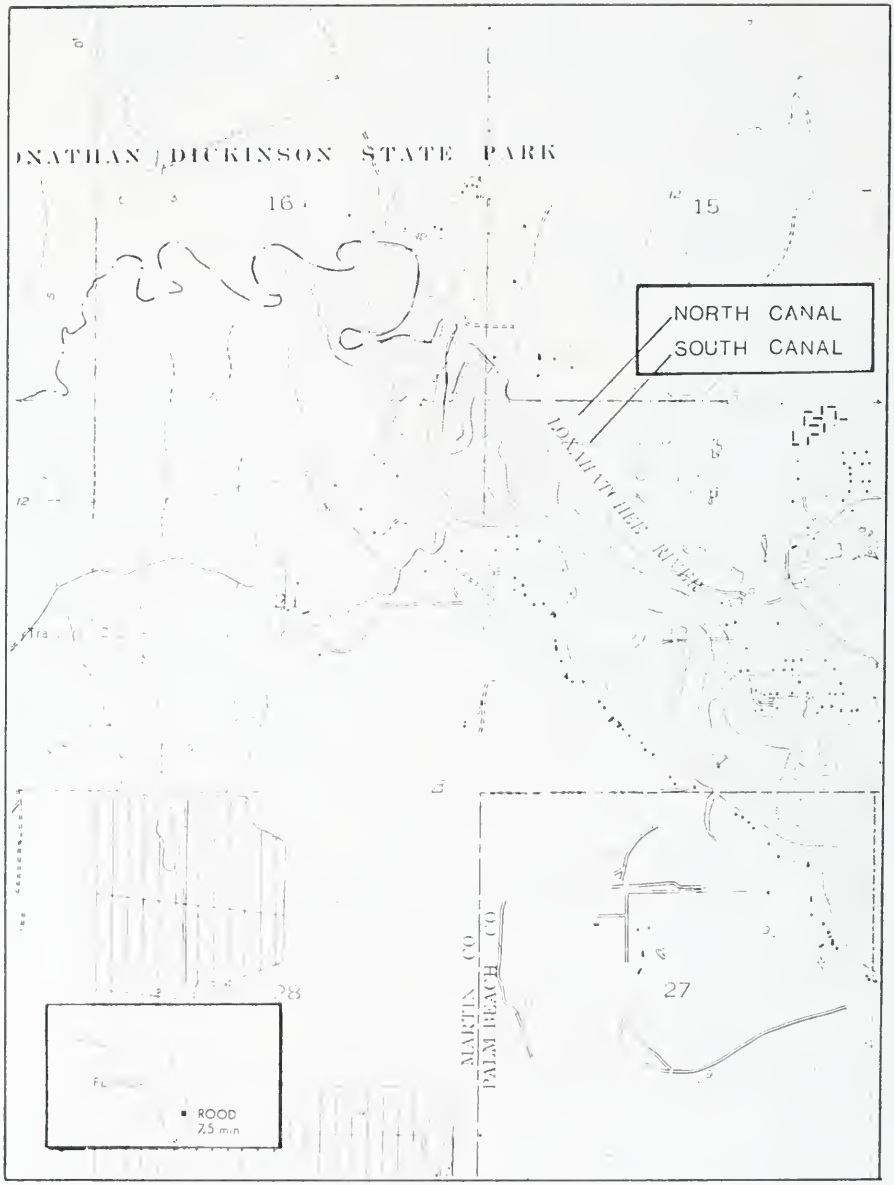


Figure 2.14 - Topographic (7 1/2' quadrangle) Map for Loxahatchee River Canals.

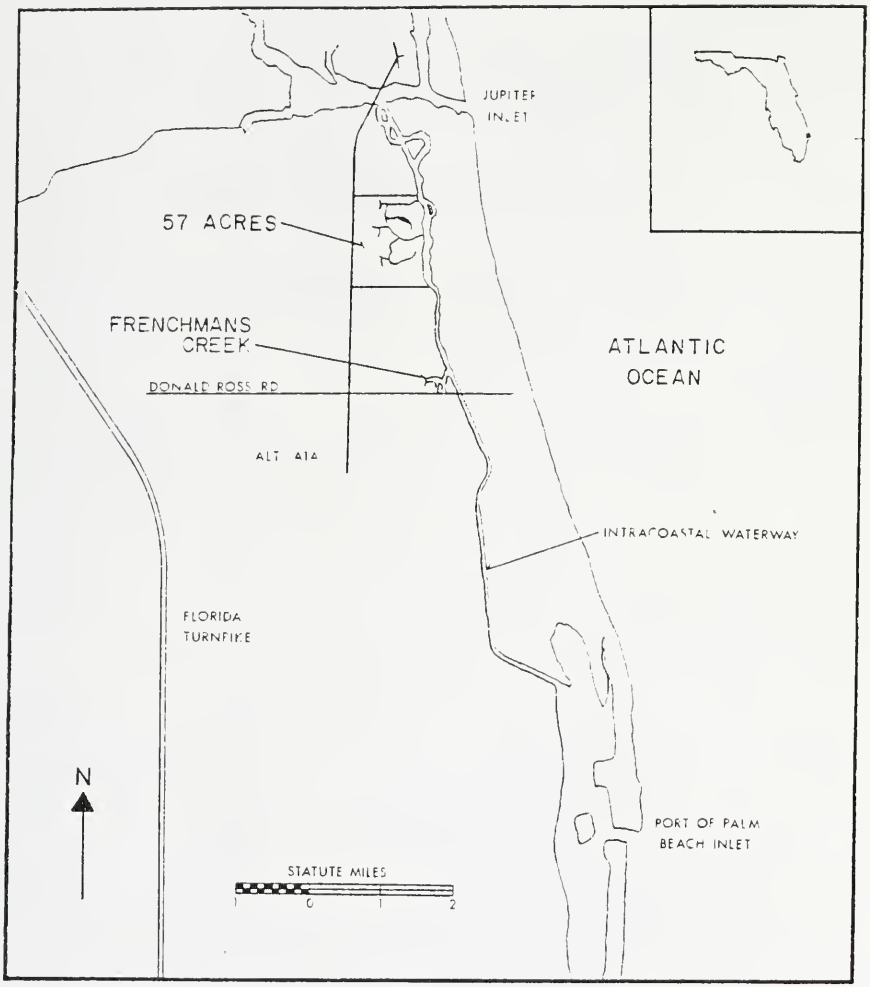


Figure 2.15 - Location Map for Frenchman's Canal and 57 Acres Canal Sites.

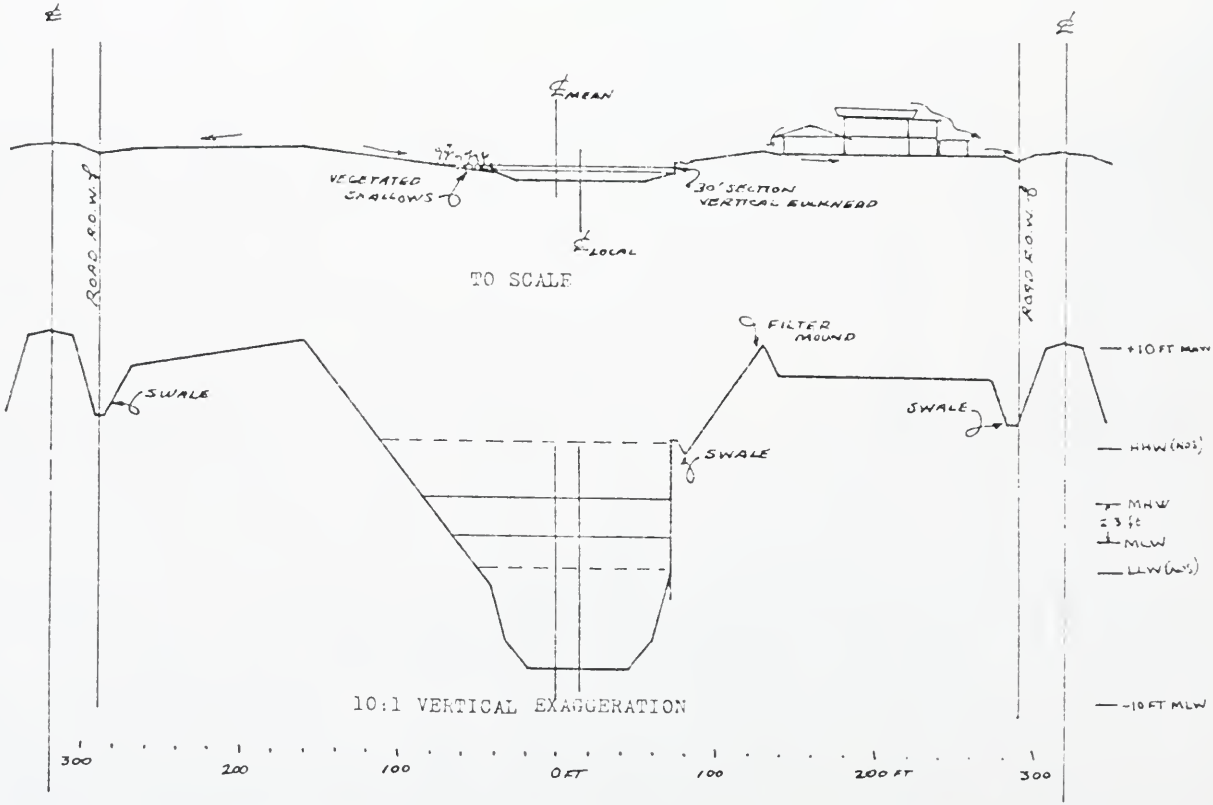


Figure 2.16 - Typical Bank Section Across Canal Showing Filter Mound and Swale, From 57 Acres Canal Design (Source: Snyder, 1976, p. 78).

CHAPTER 3

THE CANAL DESIGN PROBLEM

A canal design is a set of specifications describing the desired appearance and operational characteristics of a residential canal development. It is a plan with sufficient detail concerning canal layout and geometry, canal network operating characteristics, arrangements for handling waste and storm water on the site and in the immediate vicinity of the canal network, marina design and other details, to enable the detailed construction specifications to be drawn up by the developer's engineers. 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 facilities, utilities, and other common features of residential developments.

The design necessarily will be a compromise among:

1. the regulations imposed by federal, state and local agencies,
2. the objectives of, and economic constraints on, the developer,
3. the technical considerations which will influence the ability of the canal system to maintain its physical stability, the quality of its waters and the overall health of its natural systems.

In this chapter the overall objectives of a residential canal design will be described, and illustrated with some typical examples. The most important considerations from the developer's viewpoint are discussed, the legislative or regulatory limitations are outlined, and information on the permitting process is presented.

3.1 Overall Objectives of Canal Design

The objectives of a residential canal design may be considered to be the *qualitative decisions* which establish the character, magnitude and ultimate compatibility of the development with its surroundings. A typical overall residential canal design objective, for example, might be to create a community which can be completed and occupied within two years, for less than some specified level of investment per unit. It is important that such limitations on the overall project be clearly defined during the early planning stages, as it is often a temptation to try to design the best possible community at any cost, rather than a community that is tailored for the market and the available site characteristics. Examples of typical design objectives are given in Table 3.1.

Once established, the qualitative, overall objectives of a canal design need to be translated into specific *quantitative design criteria*. To do so, design guidelines are selected as appropriate for the site and the overall objectives, and by means of these guidelines the canal designer develops a quantitative, trial canal design. The concepts of objectives, guidelines and constraints are illustrated in Table 3.1. These terms will be more fully developed and illustrated in Chapters 4 and 5.

Design *constraints* are defined as quantitative limits or boundaries to the design. Thus, they are equivalent to the constraints used in a linear programming problem, establishing a range within which an optimal solution is to be found.

3.2 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 avoids storm damage and law suits based on inadequate construction in the future [Resource Planning Section, 1975, pp. 33-35]. Some of these costs are legislated into the design through local, state and federal regulations. This, in general, 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.

As a general rule of thumb, it is probably correct to say that 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 (Table 3.2) and environmental checklists (Table 3.3). Local planning agencies and all tiers of government are available to explain regulations and to assist in design decisions, since they would prefer that a design be well thought-out ahead to minimize future problems and adverse impacts.

3.3 Legislative Considerations

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 to be 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. It is the purpose of this chapter to put these responses into perspective by explaining their purposes, meanings, interrelationships and influence on residential canal design in the coastal zone.

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.

Table 3.4 illustrates in general the level of concern of federal, state and local agencies with regard to various categories of land use. The table is divided into upland and coastal land use. Upland land use is subdivided into three categories which reflect increasing potential for environmental impact, and thus an increasing range of governmental control. Approval for coastal land use is required, at least, at all three of the upper levels of governmental jursidiction due to the sensitivity of the environment at the shoreline.

3.3.1 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 a growing awareness of the need to restrict certain of these activities in the national interest. In particular, the intent of the EPA with regard to filling in wetlands is unambiguous:

> From a national perspective, the degradation or destruction of aquatic resource by filling operations in wetlands is considered the most severe environmental impact covered by [the rules and regulations concerning discharge of dredged or fill material in navigable waters].
> [EPA, 1975c p. 41294]

Recognizing, however, that situations may arise in which dredging and filling in a portion of the wetlands could have a net benefit, EPA further clarifies the policy of the federal government (under authority of the Federal Water Pollution Control Act Amendments of 1972, P.L. 92-500) to be as follows:

> (i.) Discharge of dredged material in wetlands may be permitted only when it can be demonstrated that the site selected is the least environmentally damaging alternative; provided, however, that the wetlands disposal site may be permitted if the applicant is able to demonstrate that other alternatives are not practicable and that the wetland disposal will not have an unacceptable adverse impact on the aquatic resources.
> ...(ii) Discharge of fill material in wetlands shall not be permitted unless the applicant clearly demonstrates the following:
> (a) the activity associated with the fill must have direct access or proximity to, or be located in, the water resources in order to fulfill its basic purpose, or that other site or construction alternatives are not practicable; and
> (b) that the proposed fill and the activity associated with it will not cause a permanent unacceptable disruption to the beneficial water quality uses of the affected aquatic ecosystem....
> [EPA, 1975c, p. 41296]

Clearly, 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, 1975c, 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 suceptible 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 acts to clarify jurisdictions, definitions and the objectives of planning activities, were issued as summarized in Table 3.5. The principal objective of these extensions and clarifications was to provide specific protection to "important resources" [Corps of Engineers, et al, 1977, p. 5] 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. One important provision of the Act provides for

> ...increased state control over federal activities. Once the federal government approves a state's management program, federal actions within or affecting a state's coastal zone must be consistent with the state's coastal management program to the maximum extent practicable.
> [Bureau Coastal Zone Planning, 1977, p. 1]

The significant federal laws regulating canal development are summarized in Table 3.5.

3.3.2 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 presently (1978) developing a comprehensive Coastal Management Program (CMP) designed to meet both state legislative requirements and the requirements of the federal CZMA. In order to meet federal requirements, the state must show through the CMP that it:

1. can control, either by state or local regulations, the uses and activities which have a direct and significant impact on coastal resources,
2. can control coastal and water uses of greater than local significance,
3. can designate and provide special management for coastal areas of particular state concern,
4. has all of the authorities needed to carry out a coastal zone management program, and that these authorities can be coordinated.

Much of the legislation required to demonstrate these capabilities is already existing in Florida.

3.3.2.1 Geographic Areas of Particular Concern

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.

3.3.2.2 Special Flood Hazard Areas

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. At the state level "there apparently are no enforceable state standards or programs for managing development activities which create public hazards due to hurricanes and flooding" [Bureau Coastal Zone Planning, 1977, p. 70]. Nonetheless, for overall guidance there is the Federal Flood Insurance Program, administered by the Dept. 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]. It would appear that this program will ultimately be recognized in the Florida Coastal Management Program.

3.3.2.3 State Legislation

The specific state legislation applicable to canal projects and related construction work is summarized in Table 3.6, 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 artifical 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.

Another important regulation of which the canal designer should be aware is the classification of the waters of Florida in terms of usage (Chapter 17-3, 403 FS, Pollution of Waters). Class 1 waters are public water supply; Class 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. Table 3.7 lists the criteria for Class III waters. The policy stated originally by the Department of Pollution Control, and adapted by the DER,

> shall be to protect water quality existing at the time these water quality standards were adapted or to upgrade or enhance water quality within the State of Florida. In any event where a new or increased source of pollution poses a possibility of degrading existing high water quality, such project development shall not be issued a Department permit until the Board is satisfied that such development will not be detrimental to the best interest of the state and necessary to its social and economic development. In administering the policy, high quality receiving waters will be protected by requiring as a

> part of the intitial project design the highest and best practicable treatment available under existing technology.
> [State of Florida, Chapter 17-3, FS, p. 7].

3.3.3 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. City and county commissions, acting through their zoning, building, planning and public safety boards can determine the use of land through five major governmental powers [Veri, et al, 1975, pp. 179-180]:

1. direct land regulation through zoning, which in theory is used to insure that land uses are properly located in relation to each other and to insure that adequate space is reserved for future development. A "planned unit development" (PUD) district is often included in zoning regulations to allow planning for a large area to include several different types of land uses, such as commercial, residential, and industrial uses. This category also includes subdivision regulations and building and related codes,
2. eminent domain, the power to appropriate land for a necessary public use,
3. spending power, to purchase lands outright for public purposes,
4. proprietary power, which subjects public land to greater control than private property by such means as reselling public land with restrictive covenants,

5. taxation power, which in addition to its use to obtain revenue, can be used as an incentive to control development.

Land use controls, and questions about their effectiveness and limitations, are complex issues which will become more and more controversial as the demand for land increases and its availability declines. 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.

Table 3.8 is a checklist for the regulatory portion of the residential canal development. 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.

3.4 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, et al, 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. In general, 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, et al, 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 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, et al, 1977, pp. 4, c-1 and c-2].

Projects which are exempted from DER permitting procedures include maintenance dredging of existing manmade 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 or seawalls, maintenance of drainage ditches, repair or replacement of existing pipes from storm water 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 jurisdiction of the Florida 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 and may be assessed an EIS preparation fee. 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 3.1 - Examples of Canal Design Objectives, Guidelines, Criteria, and Constraints.

Examples of Canal Design Objectives

Type of Community: ¼-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-year, 6-hour storm.
Magnitude of Investment: $500,000.
Time Span of Work: two years after receipt of 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 diverted to detention ponds.

Examples of Canal Design Criteria

Flushing of normal pollutant load to be less than 20 hours under normal climatic conditions.
Canal bank slopes to be 1:5.

Examples of Canal Design Constraints

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 $\geq$ 0.25.
Orientation of axis of poorly flushing channel to be within 10 degrees of prevailing summer wind direction.

Table 3.2 - The Development Process

1 INITIAL FEASIBILITY

The three significant factors to be investigated are

A. Market
- demand
- mix of land uses
- absorption rate
- required amenities in order to be competitive

B. Site
- ecological characteristics
- land area holding capacity in terms of proposed uses
- impact of surrounding land uses
- relation to surrounding ecology
- special amenity opportunities (view, vegetation bodies of water recreation access, etc., on or near site)
- access to site in relation to service area
- availability of key utilities essential to development

C. Government
- determine agencies within government to contact
- determine attitude of local government and community toward growth and toward type of development proposed
- understand local zoning and building code requirements
- determine utilities and pollution control requirements of EPA
- measure costs and benefits to community in relation to public services and facilities

Evaluate above in relation to the profit plan and decide if it is worth time, energy, and risk involved. Only after this can an intelligent decision be made as to whether or not to proceed to the next, more detailed design stage

2 THE GENERAL PLAN

The four significant factors to be investigated are

A. Preparation of general plan
- site plan layout
- building types
- utilities
- roadways
- character and appearance concept

B. Construction phasing
- determine construction phasing in terms of market (private) need (public) and funding
- prepare construction phasing plan with completion dates for each
- prepare development costs schedule
- prepare critical path for decision making approval and development

C. Environmental impact assessment
- determine impact as part of planning evaluation
- federal and state requirements
- local ordinance requirements

D. Negotiations and review of overall concept
- federal, state and local public agencies review concerning
  - zoning and plan development requirements
  - utility approvals
  - public hearings
- negotiations for financial approval

Evaluate the impact of the proposed development in relation to the area affected. Environmental assessment is a key tool to this process since it will identify needs and requirements for environmental protection and related costs

3 DETAIL DESIGN OF FIRST PHASE

The significant factors to be investigated are

A. Analysis of particular aspects of first phase of project
- market
- site
- government and community receptivity
- regional considerations
- economic analysis
- ecological impact
- planning considerations

B. Detailed site planning and design
- site grading
- utility layouts
- building prototypes orientation, arrangement, floor plans, etc
- amenity design—landscape plans, views, street furniture etc
- recreation facilities
- density limits within general plan
- determine exact type, number, and location of buildings, utilities, etc.
- community urban design—architectural, landscape, circulation, etc

C. Construction cost budgets

D. Sales or rental objectives

4 FINAL GOVERNMENTAL APPROVAL OF FIRST PHASE

A. Federal, state, local agencies review for approval
- plan modifications as necessary
- revaluation of economics

B. Preparation of covenants, etc.

5 IMPLEMENTATION

A. Final construction documents
- site design, details, etc.
- working drawings

B. Construction schedule
- phasing
- administration

C. Pollution abatement during and after construction
- noise
- air
- water

This planning procedure is repeated for each phase of development. Each phase must be evaluated in terms of sales response, user response, cost, and construction problems. Continuous evaluation of environmental engineering aesthetics efficiency should be made. Within the guidelines established in the general plan, any additional phase can be adjusted to better reflect the current needs and desires of the users and surrounding community

Source: Veri, et al, 1975, pp. 142-143.

Table 3.3 - Environmental Checklist Worksheet: A Selected List

	Site Selection and Evaluation	Site Planning	Building Design	Construction	Management
A AIR QUALITY AND CLIMATE					
Pollution					
Carbon monoxide	•	•	•		•
Hydrocarbons	•	•	•		•
Oxides of nitrogen	•	•	•		•
Sulfur oxides	•	•	•		•
Particulate matter	•	•	•	•	•
Odors and particles	•	•	•	•	•
Clarity and aesthetic quality					
Weather and climate (orientation, etc.)	•	•	•		
B WATER RESOURCES					
All types of water					
Freshwater availability					
Domestic water	•	•	•		•
Irrigation water					
Quality of water					
Physical	•	•	•	•	•
Chemical	•	•	•	•	•
Bacteria content	•	•	•	•	•
Aesthetics	•	•	•		•
Sewage treatment system	•	•			•
Water storage					
Drainage and runoff		•		•	•
Percolation and flood plains	•	•		•	•
Groundwater	•	•			•
Construction on aquifer recharge area	•	•	•	•	

	Site Selection and Evaluation	Site Planning	Building Design	Construction	Management
C GEOLOGY, TOPOGRAPHY, LAND FORMS, AND SOILS					
Soil and geologic stability	•	•	•		
Erosion and sedimentation		•		•	•
Aesthetic preservation	•	•	•	•	•
Pollution					
Pesticide		•		•	•
Herbicide		•		•	•
Toxic materials		•		•	•
Top soil removal		•		•	
D WILDLIFE AND HABITAT					
Habitats and migration areas	•	•	•		•
Specific wildlife	•	•	•	•	•
Fish contamination		•		•	•
E OTHER RESOURCES					
Vegetation					
Destruction during construction		•	•	•	
Replacement value		•	•	•	•
Scenic areas and parks	•	•	•		•
Wilderness areas and forests	•	•			•
Historic sites or structures	•	•			•
Archaeological sites	•	•			•
F LAND USE (Adjacent)					
Conservation and preservation areas	•	•	•		
Recreational	•	•	•		

	Site Selection and Evaluation	Site Planning	Building Design	Construction	Management
Agricultural	•	•	•		
Residential	•	•	•		
Commercial	•	•	•		
Industrial	•	•	•		
Institutional	•	•	•		
Division or disruption of an established community, e.g., dividing residential from recreational or disrupting planned development	•	•	•		
G SERVICE SYSTEMS FACILITIES					
Public schools	•	•			
Police facilities	•	•			
Fire protection facilities	•	•	•		
Health facilities	•	•			
Water systems	•	•	•		
Power systems	•	•	•		
Sewerage systems	•	•	•		
Refuse disposal systems	•	•	•		
Other public facilities and utilities	•	•			
H TRANSPORTATION SYSTEMS					
Air transportation	•	•	•		
Land transportation	•	•			
Water transportation	•	•			
Effect upon air quality	•	•	•		•
Effect upon water quality	•	•	•		•

	Site Selection and Evaluation	Site Planning	Building Design	Construction	Management
Adequate access to site	•	•			
I NOISE LEVELS AND VIBRATION					
On-site noise and vibration	•	•	•	•	•
Off-site noise and vibration	•	•	•		•
Construction in airport noise zone	•	•	•		•
Construction near high volume highway traffic	•	•	•		•
J SAFETY					
Fire safety	•	•	•	•	
Construction in flood plain	•	•	•	•	
Disease control, rodent control	•	•		•	•
Radiation and air pollution levels	•	•	•	•	•
Construction under high voltage lines or over high pressure gas lines	•			•	
K THE ECONOMY OF THE AREA AND THE SURROUNDING REGION					
Tax revenues generated	•	•			
Inside the project area	•	•			
Outside the project area	•	•			
Cost and availability of public services					
Inside the project area	•	•	•		
Outside the project area	•	•	•		

Table 3.3 - Continued.

Employment				•	•		Community structure					
[illegible]	•						[illegible]	•	•	•		
[illegible]	•						[illegible]					
[illegible]	•						[illegible]	•	•			
[illegible]	•						[illegible]	•	•	•		
L SOCIAL REALITIES							[illegible]	•	•			
Population (Market Area)							Employment opportunity	•	•	•	•	
Total [illegible]	•	•	•				Educational opportunity	•				
Density			•				Physical, mental, and emotional health	•	•	•		
Distribution	•	•	•				Personal safety	•	•	•	•	
Ethnic composition	•	•	•									

• primary importance
secondary importance

Source: Veri, et al, 1975, pp. 162-164.

Table 3.4 - Government Decision Making

GOVERNMENT LEVEL	UPLAND USE			COASTAL LAND USE	
	Routine Land Use	Land Use of Regional Impact	Environmentally Sensitive Land Use	Submerged Land Use	Marine Water Use
Local	●	●	●		
Regional		●	●	●	●
State		●	●	●	●
Federal			●	●	●

● Need for clearance from designated authority level

Depending upon type of activity, clearance may not be needed

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Source: Veri, et al, 1975, p. 169.

Table 3.5 - Partial List of Federal Laws Regulating Canal Development

Statute	Effect
Rivers & Harbors Act of 1899	All work in, connecting to or affecting navigable waters.
PL91-190 National Environmental Policy Act of 1969	Coordination of Federal Activities.
PL91-224 Environmental Quality Improvement Act of 1970	Established the Office of Environmental Quality and the CEQ.
PL92-500 Federal Water Pollution Control Act of 1972	Discharge of pollutants - EPA Discharge of Dredged Materials into Navigable Waters C of E.
PL92-583 Coastal Zone Management Act of 1972	Federal Standards for local control of Coastal Zone Activities.
PL93-234 National Floodplain Insurance Program of 1973	Federal flood insurance guarantees to qualifying local governments.
Federal Regulations	
Environmental Protection Agency - Federal Register 5SEP75	Discharge of dredged materials.
U.S. Army Corps of Engineers* - Federal Register 19JUL77	Regulatory program of C of E in "Waters of the United States".

*[illegible]

Source: Snyder Oceanography Services, 1978.

Table 3.6 - Authorities to be Utilized and State Agencies Involved in Activities Related to Canal Development.

Uses/ Activities Subject to Management	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 Agencie:

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 - DBR: 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 of 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

Source: Bureau Coastal Zone Planning, "The Florida Coastal Management Program. Workshop Draft," Tallahassee, FL, October 1977.

Table 3.7 - State of Florida Criteria for Class III Waters.

Class III Waters: "Recreation" - Suitable for Propagation and Management of Fish and Wildlife.

"The following criteria are for classification of waters to be used for recreational purposes, including such body contact activities as swimming and water skiing; and for the maintenance of a well-balanced fish and wildlife population. All surface waters within and coastal waters contiguous to these basins, including off-shore waters, not otherwise classified shall be classified as Class III; however, waters of the open ocean shall be maintained at a dissolved oxygen of not less than five (5.0) ml/l. Streams specifically listed in Section 17.3.31 by a separate listing designated as "Special Stream Classification" shall similarly be maintained at a minimum dissolved oxygen level of five (5.0) ml/l.

VARIABLE	CRITERION
Wastes	"sewage, industrial wastes, or other wastes - any industrial waste or other wastes shall be effectively treated by the latest modern technological advances as approved by the regulatory agency."
pH	"of receiving waters shall not be caused to vary more than 1.0 unit above or below normal pH of the waters; and lower value shall be not less than 6.0, and upper value not more than 8.5. In cases where pH may be due to natural background or causes outside limits stated above, approval of the regulatory agency shall be secured prior to introducing such material in waters of the state."
Dissolved Oxygen	"the concentration in all surface waters shall not average less than 5 mg/l in a 24 hour period and never less than 4 mg/l. Normal daily and seasonal fluctuations above these levels shall be maintained. Dissolved oxygen concentrations in estuaries and tidal tributaries shall not be less than 4.0 mg/l except in naturally dystrophic waters. In those cases where background information indicates prior existence under unpolluted conditions of lower values than required above, lower limits may be utilized after approval by the regulatory authority. Sampling shall be performed according to the methods approved by the Florida Pollution Control Board."
Bacteriological	"in those waters designated for body contact recreation, fecal coliform shall not exceed a monthly average of 200 per 100 ml of sample, nor exceed 400 fecal coliform per 100 ml of sample in 10 percent of the samples, nor exceed 800 fecal coliform on any one day, nor exceed a total coliform count of 1,000 per 100 ml as a monthly average, nor exceed 1,000 per 100 ml in more than 20 percent of the samples examined during any month; nor exceed 2,400 per 100 ml on any day. In those waters not normally used for body contact recreation, fecal coliform shall not exceed a monthly average of 500 per 100 ml of sample, nor exceed 750 fecal coliform per 100 ml of sample in 10 percent of the samples. Monthly averages shall be expressed as geometric means based on a minimum of 10 samples taken over a 30 day period. MPN of MF counts may be utilized."
Toxic Substances	"free from substances attributable to municipal, industrial, agricultural or other discharges in concentrations or combinations which are toxic or harmful to humans, animal or aquatic life."
Deleterious Materials	"free from materials attributable to municipal, industrial, agricultural or other discharges producing color, odor or other conditions in such degree as to create a nuisance."
Turbidity	"shall not exceed fifty (50) Jackson units as related to standard candle turbidimeter above background."

Source: Chapter 17-3, F.S., Pollution of Waters, Amended 7-3-73.

Table 3.8 - 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 and 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, and 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 and 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

CHAPTER 4

CANAL DESIGN OBJECTIVES, GUIDELINES, CRITERIA AND CONSTRAINTS

Canal design objectives, as defined in Chapter 3, are the qualitative guidelines under which the canal designer and the developer cooperate to produce a set of quantitative design criteria. In this chapter the general alternatives which are available in the initial planning stages of a canal design in terms of design objectives and design criteria will be discussed.

4.1 Formulation of Design Objectives

The process of fitting a development involving substantial construction into the natural environment so that the two function together harmoniously is a process which requires a geat 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 the state and federal 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. One way in which these many factors can be balanced and considered on a common basis is shown in Table 4.1. Here the various considerations are categorized in accordance with the individual or group responsible for

making decisions, and the basic questions each must satisfy. The common denominator is cost, both environmental and economic. One of the difficulties with such a decision-making process, as stated in the footnote to the table, is that a satisfactory balance is very difficult to find if the problem is considered as a whole. The usual procedure is for each group to consider the questions that pertain to its sphere of influence and to attempt to maximize the benefits and minimize the costs within that subset of questions. In such a process tradeoffs are common, and the final agreement often depends more on individual negotiating ability than on objective analysis.

Whether a particular site inspires a development concept, or a development concept initiates the search for a suitable site, the characteristics of the site limit the canal design plan. But while a site constrains a development in certain ways, it also provides opportunities which may be unique. Thus, the statement of design objectives, which of necessity relates to a particular site, is an initial formulation, in general terms, of the development concept which the developer would like to implement. These design objectives provide a starting point for the canal designer, who will develop from them a quantitative set of design criteria which 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 4.2 some principal canal design objectives are presented with some of the decisions that must be made to define these objectives.

Defining the type of community is one of the most important of these decisions. It is based on the availability of investment capital as much as on any other factor; conceivably, limited capital could result in a suboptimal design, considering the opportunities inherent in the site. Part of the canal designer's task is to consider and suggest alternatives which could enhance the development and make better use of the opportunities available.

In defining the type of community which best utilizes the developer's resources, at least four aspects relative to canal design can be identified, which should be examined. The first is the market potential for the units after the development has been completed. This is an important decision because by means of the lot size it determines, the extent and shape of the canal network, the storm water and waste water drainage systems, and all of the other physical characteristics of the final design.

The character or appearance of a site, and its aesthetic appeal, determine to a great extent the value that will be attached to a unit by the people who are attracted to it. The degree of access to the water also relates to the image of the development, as does its navigability. Provisions for docking boats at each lot generally implies a need for greater canal widths to accommodate the expected traffic, and the maximum size of boats to be accommodated in the canal system will affect the minimum depths of channels and the size of the boat basin for maneuvering. An indication of a reasonable design boat size is contained in EPA's statement that there were more than 297,000 boats registered in the State of Florida in 1974, 94 percent measuring 26 ft or less. [EPA, 1975b, p. 7].

The source of water supply must be from off site unless saltwater intrusion is not a problem, and the aquifer is sufficiently shallow. If onsite water supply is desired, the water must meet the state specifications for Class I waters and provision will have to be made in the drainage plan and the septic tank plan to avoid groundwater contamination. The cost of offsite water may be economical in comparison to the cost of pumping freshwater on site, depending on the difficulty of obtaining, handling and monitoring an onsite source.

Waste water may be handled either onsite or offsite, depending again on the characteristics of the site and the canal, and the number of units in the development. Septic tanks and related devices for underground disposal of household sewage can become major potential sources of pollution when located near the banks of canals. If the natural watertable is high, the liquid portion of the waste can saturate the soil and then run off to the canal. In areas which are subject to flood tides or storm tides, which can also raise the water-table high enough to affect septic tanks, it may be advisable to decide in favor of a central waste water treatment system, or offsite treatment.

EPA has documented extensive studies on septic tank leaching in residential canal developments [EPA, 1975b, pp. 159-186]. In their report on finger-fill canal studies it is observed that septic tank/ sorption 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, 1975b, 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, 1975b, 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, 1975b, pp. 178-179].

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

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 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 [1975b, pp. 3, 9, 159-186], and U. S. Public Health Service [1967].

Rain storms create pollution 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. The best solution, again, is usually onsite handling, which is accomplished by using the topography of the site, supplemented by swales leading to detention ponds. 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 these can also be used as parks or recreation areas during normal conditions. The alternative is to provide the necessary drainage systems, collection facilities, pipes and pumps for moving storm water offsite. Storm water and waste water systems can also be combined, although this is an inefficient and costly way to recycle water.

The advantages of sloping canal banks over bulkheaded banks were briefly mentioned in Section 2.6. The decision to be made here is one which will affect the final appearance, operating characteristics and cost of the canal system to a significant degree. If sloping banks are used to ensure channel stability, to provide a larger tidal prism, and to provide greater water surface area for improved dissolved oxygen content and improved lateral navigability for small boats, less area will be available for the development of lots. However, the resulting improvement in flushing characteristics, bank stability and aesthetics will ensure significantly fewer maintenance costs and higher property values, and in addition will be much easier to permit than a system with vertical bulkheads.

In concluding this section on the identification of design objectives, which is equivalent to the first stage in the planning process, it is noted that in selecting and/or evaluating a site there are several important factors to consider:

1. the opportunities and constraints of the site in relation to the proposed development, such as elevation, hazards, soil types, water table elevation, drainage characteristics, etc.,

2. the impact upon the community at large and especially the adjacent neighborhood, inasmuch as the impact will vary with the size of the development and its intensity.
[Veri, et al, 1975, p. 130].

The overall objective of the planning process is to avoid exceeding the environmental tolerances of the site. In particular, regarding canals, the objective is to avoid exceeding the capacity of the waterway and the receiving waterbody to assimilate pollutants. In Chapter 5 the assessment of such site characteristics will be considered.

4.2 Design Guidelines, Criteria and Constraints

Design *guidelines* may be defined as qualitative statements which serve to guide the designer toward meeting an established design objective. For example, the statement that "any absorption field in a system utilizing septic tank systems must be set back at least 150 feet from the annual high-water line" may be considered as a guideline. A design *criterion*, on the other hand, is defined herein as a statement which gives the range (or the definite value, if no range is permissable) of a variable which will be used in a specific design. Most design criteria will follow from design guidelines. Thus, the septic tank guidelines just given might be used by the designer to establish the criterion that "septic system absorption fields will be located 200 feet from any canal at mean high water."

Design *constraints* are another form of design criteria, but they are used in this development in the same context as in an optimization problem. If a design guideline establishes a range for a certain variable, the designer may prefer to use that range 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 ft, he may decide to find the depth or combination of depths within 4 to 8 ft which maximizes the flushing rate of the canal.

Suggested design guidelines for projects which affect the environment are available from a variety of sources. In order to show the range of guidelines applicable to residential canal design which have already been published, one example of a set of guidelines has been selected from each of six different decision-making areas which have an interest in coastal development. These sets are listed in Table 4.3. This table also lists the principal viewpoint of the institution or agency which produced the guidelines, and the subsequent table in which each is quoted or summarized.

The first of the six sets of guidelines (Table 4.4) consists of suggestions by governmental agencies involved in evaluating development projects. These sets of statements were obtained in response to a letter written by R. M. Snyder, a canal designer in Florida, to the Jacksonville District of the U. S. Army Corps of Engineers (Corps), the Fish and Wildlife Service (FWS), U. S. Department of Interior, and the Florida Department of Natural Resources (DNR) asking for comments on a preliminary list of canal design considerations. Two replies to this inquiry were quite specific, and are excerpted in Table 4.4. The point made by Col. Goode of the Corps, that artifical waterways should more closely follow the examples designed by nature, is borne out in a variety of ways by the research results of the canal design project. The comments given by Mr. Vaughn of the FWS, which reflect the biologist's point of view, are in all cases supported by the hydrodynamic viewpoint which has prevailed in the canal design project.

The second set of guidelines (Table 4.5) has been taken from the booklet accompanying the forms for joint permit application for dredge and fill projects in Florida [Corps of Engineers, et al, 1977]. These

statements of policy by the federal and state agencies responsible for evaluating projects which may have an environmental impact, and for issuing permits which allow construction in the coastal area are, as they should be, relatively nonspecific. The Corps statement makes clear that they are responsive to national concern and the public interest, while the Florida DNR emphasizes that they are sensitive to plans which may have destructive effects on important parts of the environment. Additional guidelines may be obtained from Chapter 253, Florida Statutes, Land Acquisition Trust Fund (Title XVII, Public Lands and Property), having to do with applications for and restrictions on filling land and dredging; Chapter 403; Florida Statutes, Environmental Control (The Florida Air and Water Pollution Control Act), having to do with matters relating to pollution; and Chapter 17-4, Florida Administrative Code, Rules of the Department of Environmental Regulation, having to do with permits.

The third set of guidelines (Table 4.6) represents an altogether different viewpoint, that of the construction industry and associated planners [Veri, A.R., et al, 1975]. Written to provide guidelines for development in South Florida, it presents a list of "constraints" and "opportunities" for each of the five types of ecosystems common to that area. The majority of these guidelines are so general, however, that they may be applied directly to residential canal planning anywhere along the southeast coast and the Gulf of Mexico. These guidelines also support the concept of designing with nature, not against it.

The fourth set of guidelines (Table 4.7) from Clark [1977], includes detailed explanations for each of the suggested guidelines. The common theme in this set is to minimize the impact on the environment

and to avoid designs which cannot survive under natural conditions.

The fifth set of guidelines (Table 4.8) represents the planners' point of view as well as that of the Government of Georgia [Resource Planning Section, 1975]. It, too, strongly recommends that natural conditions at the site should guide the design, and that there are many common sense, economic ways to design with nature.

The sixth set of guidelines (Table 4.9) was developed by R. M. Snyder [1976] in 1975 near the beginning of the canal design project. These guidelines show the engineer/hydrodynamicist's approach to ensuring that a canal will have good water quality. It is based on the same framework as Table 1.2, which presents the traditional approach to canal design. It is not surprising that a rational plan for design based on sound engineering and hydrodynamic principles arrives at the same conclusions as the regulatory, biological, developmental, conservation and planning viewpoints. They are all trying to fulfill the same, interrelated objectives with the same natural resources.

Table 4.1 - Balancing Environmental and Economic Costs and Benefits

DECISION MAKING INSTITUTION	QUESTIONS TO SATISFY
Developer	Market feasibility Land and development costs Scheduling Cash flow Delays Materials availability and supply Marketing and promotion and others
Government	Zoning Environmental impact Community amenities Providing timely services and utilities Tax gain/loss Building codes and others
Community	Change in character Tax burden Visual characteristics Loss of amenities Social compatibility Job resource and others

NOTE Each sector is primarily concerned with maximizing the benefits while minimizing the costs, a balance that can only be achieved if each institution is considered separately. If, however, all the questions are reviewed collectively, the task of satisfying each to the benefit of its institution and to the satisfaction of the other institutions becomes complex. Certainly, trade-offs will be made between the groups to reach an optimum situation, i.e., that point at which most benefits are realized at least cost to the mutual satisfaction of all parties concerned.

Providing guidelines for the achievement of this optimum situation is the primary objective of this book. Although it is recognized that not all the land area of South Florida should be developed in equal intensity—and that some areas should not be developed at all—suitable areas must be identified and rational alternative methods for building and development be followed.

Source: Veri, et al, 1975, p. 138.

Table 4.2 - Principal Design Objectives Relating Particularly to Residential Canal Design.

Objectives	Considerations	Range of Decision
Define investment parameters	Level of investment	Equity required
	Duration of investment	Loan restrictions
	Time span of work	Time for obtaining permits Time after permits received
Define type of community	Potential market characteristics	Lot size: large, small, or mix Lot density Lot cost
	Character (appearance) of the community and its aesthetic appeal	Urban vs. rural Developed area/open area ratio: specific areas reserved degree of preservation of shoreline
	Degree of access to water	Access from: each lot or marina only Both individual docks and marina
	Navigability of canals and basin(s)	Tidal entrance(s): number location(s) Canal depths and widths desired Marina depth and shape desired
Source of water supply		Groundwater (onsite) Offsite supply
Method for handling waste water	Sewage	Onsite: septic tanks or central treatment
	Storm	Offsite disposal or Detention basin arrangement

Table 4.3 - Decision-Making Agencies or Institutions with an Interest in Coastal Development.

Decision-Making Area	Agency or Institution	Viewpoint	Design Guidelines
Federal Government	Environmental Protection Agency U.S. Fish and Wildlife Service	regulatory biological	Table 4.4
Federal and State Government	U.S. Army Corps of Engineers Florida Department of Natural Resources	regulatory regulatory	Table 4.5
Construction Industry	Associated General Contractors	development	Table 4.6
Conservation	The Conservation Foundation	conservation	Table 4.7
Regional Planners and State Government	Georgia Coastal Zone Management Technical Committee Georgia Department of Natural Resources	planning regulatory	Table 4.8
Private Industry	Canal Designer	rational engineering design	Table 4.9

Table 4.4 - Summary of Comments Relative to Canal Design Guidelines Received from Federal Agencies in Response to a Request (December 4, 1974) for Comments on the Need for Research on Canal Performance.

Corps of Engineers, U.S. Army

"Normally canals and/or artificial waterbodies are prone to environmental problems unless special (usually expensive) precautions are taken to insure circulation and means for removal of suspended and dissolved materials are provided. Nature provides removal of materials by various methods such as vegetation and cyclic flows thus allowing oxidation and aerobic decomposition to accelerate the cleanup process. It would seem to us that techniques could be developed to design and maintain artificial canals by similar means that nature provides for natural waterways."

Source: Goode, B.H., Chief, Regulatory Branch, December 17, 1974.

Fish and Wildlife Service, U.S. Department of the Interior

A. "VALUE OF CANALS

1. "Aesthetic"

a. curved canals: "Orthodox concepts of design aesthetics would, in general, favor curved or serpentine configurations over the conventional arrow-straight form taken by most residential canals in Florida and elsewhere in the southeast."

b. canal banks: "Stabilization of canal banks with conventional bulkheads, in our view, conveys a strong impression of artificiality to canal design, in addition to the ecological disadvantages such structures usually present. Sloping riprap is, in our view, generally preferable to vertical bulkheading, both on aesthetic grounds and because of the additional surface area that it provides for attached organisms."

2. Fishing: "Proper canal design can result in provision of some fishery habitat and thus augment recreational fishing opportunity to some degree. The degree to which these sport fishery benefits are available will depend, in part, upon the type of shoreline development present. A dense concentration of docks and boats along the shoreline inhibits fishermen access from the water."

3. Ecological: "Canals cut into upland areas formerly supporting insignificant or only moderately productive terrestrial fish and wildlife habitat may, with proper design, become more productive than the pre-existing upland. Proper design will include considerations of depth, flushing, cross-sectional configuration, shoreline stabilization methodology, land runoff control, and other characteristics as discussed in some detail below."

a. canal bottoms: "Unevenness of canal bottoms often [occurs] as a result of hydraulic dredging. Deep, poorly flushed bottom depressions act as nutrient traps and serve as foci for anaerobic decomposition."

b. canal depth: "Excessive overall depth [precludes] light penetration to deeper waters. Under such conditions, rooted vegetation cannot grow on the bottom. The entire bottom of such canals may function as nutrient traps."

c. sills and salinity: [There is often a] "disparity between deep canal waters and shallower receiving waters. In such cases a sill is present at the canal mouth, impeding tidal exchange beyond the depth of the receiving waters. In addition, the deeper saline waters brought into the upper reaches of the canals by seasonally high tidal action becomes trapped beneath an overlying stratum of lower salinity water. The trapped saline layer becomes stagnant and depleted in dissolved oxygen."

d. water quality: [There have been cases of] "degradation of water quality through influx of upland street and lawn drainage, i.e., urban runoff. Ideally, these contaminants should be collected and routed away from any canals having poor flushing characteristics."

Table 4.4 - Continued.

B. "CONDITIONS NOT CONDUCIVE TO GOOD CANAL DESIGN

1. "Ecological: We do not have any evidence to demonstrate that any artificial canal yet devised by existing technology will equal the biological productivity of coastal tidal marshes or mangrove swamps. Consequently, the presence of such wetlands on a given site may be generally regarded as contraindicating, on biological grounds, the construction of any canal system. Stated more plainly, a canal design requiring dredging of productive wetlands is a biologically undesirable canal design. This does not mean that no canal should ever be constructed in wetlands. Public interest could require otherwise. In such an instance, design should be such as to minimize adverse ecological impact. The following is a partial list of factors we believe to be inimical to good design:"

a. canal depths: "Excavation to depths in excess of the photic zone (zone of light penetration), in excess of the depth of receiving waters (see [Section 3b] above), or in excess of the depth required for adequate flushing and exchange."

b. canal sides: "Vertical, or 'box-cut' canal banks. This configuration reduces availability of shallow water fish and wildlife habitat along the shoreline."

c. canal bottoms: "Irregularly contoured canal bottoms, as discussed [in Section 3a] above."

d. runoff: "Grading of adjacent land to permit direct discharge of polluted runoff into canals. This is a fatal defect of many canal systems. Retention ponds or swales should be provided to handle urban or agricultural runoff. Direct input to canals should be held to the absolute minimum."

e. dead-ends: "'Dead-end' design. The ecological liabilities of this type of canal design are so well known that we will not recite them here. Some shallow, relatively short dead-end canals may maintain satisfactory water quality. However, virtually any flow-through canal is ecologically superior to a dead-end canal of comparable length, depth, etc."

f. septic tanks: "Siting of canals adjacent to septic tanks and drain fields. Recent studies by the U.S. Environmental Protection Agency (EPA) have documented that septic tank drain field effluent can leach into adjacent canals with astonishing rapidity. The implications of this upon water quality are obvious."

g. entrances: "Placement of canal openings where boat traffic to and from the canals will cause erosion of adjacent or opposite natural shorelines."

h. dredging into the aquifer: "Incision into a shallow freshwater aquifer... accelerates loss of freshwater to sea and/or intrusion of saline water into the freshwater supplies of coastal areas."

i. dredge spoil: "Failure to set aside adequate disposal sites to accomodate maintenance dredging spoil. Ideally, disposal sites should be located in upland areas of low biological productivity."

C. "A METHODICAL APPROACH TO GOOD CANAL DESIGN"

1. "Hydrology of Site and Surrounding Area"

a. channel routing: "In order to locate a canal so as to minimize environmental degradation, the canal route should be selected so as to avoid the most biologically productive areas, either wetland or upland. Segmentation of habitat should be discouraged by locating the canal route near the margins of the least productive vegetative zones but generally not at the ecotonal boundaries between zones."

b. tidal differential: "One hydrology feature that should be incorporated into canal designs where possible is the linkage of opposite ends of the canal with waters having sufficient differential in tidal periodicities to force a substantial flow through the canal at high and low tides. Unfortunately, few sites offer such an advantage to planners. Islands lying adjacent to the ocean on one side and forming semienclosed bays on the opposite side are particularly conducive to this strategy, due to the tidal differential created by the tidal dampening effect of the islands upon the waters of the bays."

Table 4.4 - Continued.

2. Biology of Adjacent Waters. The entrances of canals and their associated offshore channels or entrance routes should be situated and marked so as to encourage navigation interests to utilize the least destructive offshore approach to canal entrances. Boat traffic across shallow grass flats should especially be discouraged. Canal openings to adjacent water should also be so located as to minimize the extent of channel dredging across shallow productive bottoms."

3. "Deposition and Erosion: With proper contouring and stabilization of canal banks, erosion can be adequately controlled. Again, we emphasize our preference for riprap as opposed to conventional bulkheading."

   "Depostion of sediments (and the accompanying necessity of maintenance dredging) is common to poorly flushed systems that accumulate organic materials and other particulate matter carried by runoff waters. Some sediments derive from canal bank erosion. This particular problem is also avoided by adequate bank stabilization or contouring. The optimum slope of canal banks for erosion control varies with soil type, water velocity, and drainage characteristics, and is a determination that we feel should be made by engineers, not biologists."

4. "Winds and Other Physical Parameters: Where significant prevailing winds are present, canal orientation should be such as to favor the wind-driven circulation of water from canals to receiving waters. Orientation of dead-end canals to permit wind-driven waters to be driven into the canals may result in an influx of floating seagrasses or other floating plant materials that enter canals, decay, and increase the biological oxygen demand of canal waters and sediments. This observation also applies to boat basins."

   "Consequently, we recommend an orientation that permits the prevailing wind to sweep across the interior of the boast basin and toward the receiving waters beyond."

5. "Entrance Design: The locations of canal entrances should be selected to minimize destruction of fish and wildlife habitat. This is generally accomplished by siting the canal entrance where it will traverse the smallest amount of productive wetland vegetation or shallow productive bottom substrate. However, if the canal opens to an offshore channel, the combined impact of canal and channel routes must be considered in minimizing damages."

   "If entrances are to be structurally stabilized, sloping riprap is generally preferred, on biological grounds, in favor of vertical surfaces, for bulkheads or groins. Either means of shoreline stabilization should be installed so as not to cause erosion in adjacent areas by wave reflection or interruption of longshore sand transport."

Source: Vaughn, R.R., Regional Director, Fish and Wildlife Service; excerpts from a letter to R.M. Snyder, March 14, 1975.

Table 4.5 - Design Guidelines for Dredge-Fill Stuctures, Established by U.S. Army Corps of Engineers and Florida Department of Environmental Regulation.

"(1) *Federal:* The Corps' decision whether to issue a permit will be based on an evaluation of the probable impact of the proposed activity on the public interest. That decision will reflect the national concern for both protection and utilization of important resources. The benefit which reasonably may be expected to accrue from the proposal must be balanced against its reasonably foreseeable detriments. All factors which may be relevant to the proposal will be considered. Among those are conservation, economics, aesthetics, general environmental concerns, historic values, fish and wildlife values, flood damage prevention, land use classification, navigation, recreation, water supply, water quality and, in general, the needs and welfare of the people. It is emphasized that if a proposed activity is to be performed in valuable wetlands, the Corps will evaluate it to determine whether it is a necessary alteration, and the unnecessary alteration or destruction of these wetlands will be discouraged as being contrary to the public interest. In determining whether the alteration is necessary, the Corps will primarily consider whether the proposed activity is dependent on the wetland resource and whether alternatives are practical."

"(2) *State:* DER will evaluate the potential impact of the proposed project on the waters of the state. In assessing this impact DER will determine for the purpose of a permit pursuant to Chapter 253, F.S., if the project will be a 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 oyster beds, clam beds or marine productivity including, but not limited to, destruction of natural marine habitats, grass flats suitable as nursery or feeding grounds for marine life, marine soils suitable for producing plant growth useful as nursery or feeding grounds for marine life; or for the purpose of a permit pursuant to Chapter 403, F.S., 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. For permits pursuant to Chapter 161, F.S., DNR will evaluate the functionality of the proposed construction and its compatibility with the existing coastal processes at the location of construction. An evaluation will be made of the protection afforded against coastal flooding and storm induced erosion and of the physical impact on adjacent properties. Public response to the project will be considered, which may include, but not be limited to, the restriction of public access, the effect on archaeological and historical values, and the impact on turtle nesting sites."

Source: State of Florida, "Joint Permit Application for Dredge and Fill Projects", 1977, pp. 5-6.

Table 4.6 - Design Guidelines Relative to Canal Design for Various Kinds of Ecosystems in South Florida, by Associated General Contractors.

THE COASTAL RIDGE

Constraints

1. "Generally, the coastal ridge is suitable for development without excessive constraints. Caution is needed in wetlands and the coastal fringe."

2. "Destruction of natural beach and dune can result in loss of beach to storm action. Restoration costs are extremely high."

3. "The coastal construction setback line has been set by the state at 50 ft inland from mean high water, an interim regulation, to be superceded by a setback line based on topographic, vegetative, and dynamic criteria for each county. [Now (1978) available for most counties in Florida's Coastal Zone.]

4. "Development should be concentrated on the ridge, where soil suitability is high and natural drainage is adequate. Marginal lands should be developed with extreme caution, if at all, to prevent environmental damage and to avoid future expenditures for flood and wind damage."

Opportunities

1. "Shorefront construction should be kept behind the stable dunes."

2. "No construction on the beach should be permitted."

3. "Mangrove areas that border coastal waters or tidal channels and estuaries should be preserved as buffer zones to maintain water quality and shoreline protection."

4. "Residential construction in the hurricane flood zone should have a first-floor level above the 100-year flood elevation. This can be accomplished by 'stilt' construction. Filling may occur in upland areas.

5. "Natural vegetation should be preserved wherever possible to minimize erosion, retard runoff, and maintain aesthetic values and habitat."

6. "Landscaping should utilize native species, since they require less maintenance, fertilizer, and water."

7. "Urban runoff should be directed to vegetated swales and holding ponds, rather than to curbs, gutters, and storm drains. This allows natural processes to improve water quality and allows time for aquifer recharge to occur."

8. "Sewage treatment plants should be located in areas not subject to flooding. Septic tanks are not suitable in such areas."

9. "Bulkheading should be avoided where possible, and natural systems, such as mangroves, should be preserved instead. Sloping riprap is preferable to vertical bulkheading."

THE FLORIDA KEYS

Constraints

1. "Lack of freshwater, except by aqueduct from the mainland, will be a limiting factor if the water supply cannot be expanded."

2. "Porous geologic structure and high water table permit effluent from septic tanks or seepage from solid-waste disposal sites to pollute near-shore waters, degrade recreation value, destroy reefs and other marine life. Waste disposal problems are a limiting factor that will require technologic solutions."

Table 4.6 - Continued.

3. "Linear configuration of the Keys makes central services such as water or waste treatment difficult to plan and costly to operate."

4. "Hurricane and flood hazard potential is high, since 99.6 percent of Keys land area is below the 100-year storm level. Evacuation is impractical, since U.S. 1 is the only road to the mainland."

5. "Sedimentation from runoff during construction activity is detrimental to water quality and marine life. Preventative measures should be used during construction, and cleared areas should be promptly replanted."

Opportunities

1. "Domestic freshwater supply could be augmented by cisterns to collect and store rainwater."

2. "Water-conserving plumbing fixtures are available and should be required in new construction or renovation."

3. "Private reverse-osmosis treatment plants, using water from the Floridan Aquifer, may be a solution for providing freshwater to large developments."

4. "Waste treatment is a serious problem, and a combination of solutions is necessary. Pace of development should be limited to the capacity of available services."

5. "Development planning should include selective clearing and clustering of built units to maximize preservation of natural vegetation and ground covers. Cleared areas should be planted as soon as possible to prevent runoff and sedimentation. Native plants species should be used."

6. "On-site retention of runoff must be provided."

7. "Planned unit developments, clustered to minimize site clearing, should be encouraged. Bonus densities could be given for site preservation and setback from the water's edge."

8. "Cluster developments with common marina facilities are preferable to conventional canal developments."

9. "Bulkheading should be avoided where possible, and natural systems, such as mangroves, should be preserved instead. Sloping riprap is preferable to vertical bulkheading."

SANDY FLATLANDS

Constraints

1. "When drainage is adequate there are few constraints to development. When the water table is high, however, flooding may occur during part of the year."

2. "An impermeable cemented layer of hardpan may occur at varying depths beneath the surface. This layer may cause localized ponds to develop and may present special problems for foundations."

3. "'Cap' rock may occur extensively at the surface, presenting special design problems, depending on the depth and bearing capacity of the rock."

4. "In some areas, particularly in the eastern flatlands, extensive wetlands serve as municipal water supplies. In these areas lowering of the water table by drainage for development should be undertaken only in the context of a regional water management plan."

Opportunities

1. "The sandy flatlands provide good opportunities for development where the water table is not high. The design concept for the Port La Belle community in Glades and Hendry counties is an example of good development practices fitted to a suitable site. Site constraints were considered carefully and became amenity features."

2. "The design concept for the development was based on a drainage system that imitates the natural system."

Table 4.6 - Continued.

"The basic feature is the use of interconnecting ponds and watercourses incorporated into a greenbelt network. This network brings areas of natural beauty to all parts of the development, while providing an effective storm drainage system. Greenbelt drainage areas range in width from 20 ft at back lot lines to 50 to 275 ft for watercourse right-of-way.

"Runoff is directed from street swales or overland flow into a secondary drainage system of wide, shallow swales about 3 ft deep, with a side slope less than 6:1. The swales are grassed and dry most of the time. Overgrowth or debris can be easily removed.

"The primary system is an extension of the canals and natural drainageways that originally existed on the site. The natural slought are to be kept intact, and natural watercourses are to be incorporated into the total development concept. The system consists of grassed swales connected with artificial ponds, which serve as borrow pits, stormwater storage reservoirs, ecological controls and visual amenities. Ponds will normally contain 5 to 6 ft of water.

"Control structures will be used to regulate water flow. The objective is to maintain a small rate of flow over a longer period of time. Discharge of runoff to the Caloosahatchee River is reduced, and water quality is enhanced by allowing time for debris to settle out, vegetative uptake of nutrients to occur, and seepage to replenish the aquifer and maintain the water table.

Native trees, such as oaks, will be planted along the greenbelt drainageways to help absorb nutrients. Where large existing trees occur, the street swales and drainageways are either diverted around the trees or have a steeper slope. The minimum distance from the tree trunk is 8 ft. Smaller trees are to be transplanted to other locations."

INTERIOR WETLANDS

Constraints

1. "Drainage of wetlands decreases freshwater storage capacity, both at the surface and in the aquifer. It may also allow saltwater intrusion at the coast."

2. "Alteration of wetlands results in loss of wildlife and recreation values."

3. "Filling affects water quality by destroying natural vegetation, reducing percolation, and increasing surface runoff."

4. "Impermeable surfaces, such as paved areas or buildings, can concentrate polluted runoff and contaminate the aquifer. They also reduce the area available for aquifer recharge."

5. "Improper disposal of solid and liquid waste in wetlands can contaminate the aquifer. Septic tanks are unsuitable as a method of disposal. Central treatment facilities should be located above the 100-year flood level."

6. "Development in the wetlands is subject to severe flood hazard.

7. "Deep wells into the brackish Floridan Aquifer can contaminate the shallow freshwater aquifer and surface waters if the wells are not properly cased and sealed."

8. "A depleted aquifer may ultimately be the limiting factor that will curtail urban growth in southern Florida. Therefore, wise use of this resource, maintenance of its quality, and assurance of its renewal, is of prime importance to the continued economic well-being and quality of life of the entire region."

Opportunities

1. "The criteria for delineating an area as suitable for aquifer recharge are as follows:

   a. The geologic formation must be permeable.
   b. Water must be available, either in the form of adequate rainfall or as overland flow. Ideally, both would occur.
   c. Maximum possible permeable surface must be preserved. This implies low surface coverage by impermeable materials, such as paving or buildings.

Table 4.6 - Continued.

d. The areal extent of the recharge zone must be sufficient to balance depletion of water resources from evaporation from waterbodies, runoff from canals, and urban use.

e. The area should be in relatively pristine condition. Least desirable are agricultural lands with their load of pesticides, fertilizers, and animal waste. Areas near sewage outfalls are obviously unsuitable, and most canals are polluted both by agricultural and urban runoff, as well as by sewage effluent."

2. "Designation of an area as an aquifer recharge area does not preclude development. However, careful assessment of local and regional needs for water in the future must be carefully assessed. The areal extent of necessary recharge must be evaluated to match these future needs. Then the recharge areas can be developed as long as certain criteria are met and important constraints are observed:

a. Sufficient organic soil cover must be preserved to maintain water quality. The filtering action of soil particles and the bioloqical and chemical processes at and beneath the soil surface inactivate many viruses and bacteria, including those which cause typhoid, dengue fever, and the like.

b. Before development occurs, natural drainage patterns of the site and the surrounding region should be identified and incorporated into the site design in order to maintain overland flow through the development. This objective can be achieved by design solutions, such as stilted buildings or green swale drainage systems that are imitative of natural systems. Where the natural pattern must be interrupted, culverts, bridges, and similar measures should be designed to approximate the natural water regime.

c. Landscaping should be designed to permit natural growth and attrition, rather than mowing and trimming. Low maintenance plant materials should be used to avoid the necessity for application of fertilizers and pesticides. Native vegetation is admirably suited, since it is well adapted to the South Florida environment and does not require chemical and human energy inputs for survival. Nutrient uptake by vegetation is a significant service performed by the natural environment in the maintenance of water quality. If such materials as grass clippings are allowed to return to the soil, however, these nutrients are released back to the water supply.

d. Use of exotic (nonnative) plant species in landscaping should be avoided. High maintenance of some exotics requires chemical inputs that cause water pollution. Low maintenance exotics, on the other hand, tend to escape from cultivation. They can invade and replace natural vegetative communities, descreasing the system capacity for providing environmental services. Wetlands, especially in disturbed areas, are particularly susceptible to invasion. Maintaining water quality depends on a diverse and complex system of plants, animals, and microorganisms. When exotics are introduced, a simple system results that lacks the effectiveness and buffering provided by a more diverse system."

MANGROVE AND COASTAL MARSH

Constraints

1. "Hurricane and storm flooding potential is high. Federal Flood Insurance regulations require first-floor elevations to be above the 100-year storm level."

2. "Development below mean high water is under state jurisdiction."

3. "Recent state actions have generally disapproved permit applications for development that would destroy significant areas of mangrove or coastal marsh."

4. "Soils are marls or mangrove peats, of low bearing capacity."

5. "Indiscriminate removal of vegetation and soils will degrade water quality by reducing capacity of the natural system to act as a trap for sediments and nutrients."

6. "Improper disposal of solid and liquid waste degrades water quality. Septic tanks are unsuitable as a disposal method. Central sewage facilities should be located above the 100-year flood level."

Table 4.6 - Continued.

Opportunities

1. "A combination of cluster development, elevated construction, and selective clearing affords a development strategy that is both environmentally sound and economically feasible. It preserves the coastal protection function of the natural system and assures that water quality will not be degraded."

2. "Planned unit development should be encouraged to allow necessary flexibility in urban design. Site planning and building location should be responsive to site conditions. Bonus densities for site preservation and setback from the edge of the water could provide incentives."

3. "Development planning should include selective clearing and clustering of building units to maximize preservation of natural vegetation and ground cover. Cluster development with common marina facilities are preferable to conventional canal developments."

4. "Elevated structures for coastal development solve several problems. Soil conditions in mangrove and coastal marsh areas generally require pilings, and flood criteria require the first flood to be elevated above the 100-year flood level, which in some places is 14 ft above mean sea level. These requirements, taken with environmental considerations, suggest that structures elevated on pilings, without filling beneath, would be the best design solution for these conditions."

5. "The economic advantages of elevated construction make it feasible to build. The alternatives of floating foundations, foundation walls, and/or filling to flood criteria elevations add unnecessary cost."

6. "Consideration should be given to light construction and multilevel design to take maximum advantage of the foundation system."

7. "Low-rise buildings should be located nearest the shoreline, with higher buildings located behind them on the upland. This design strategy affords many more units a view of the water and reduces the impact of development on natural coastal systems."

8. "The heavy equipment necessary for construction on pilings can cause a great deal of damage to natural vegetation on the site. Access for equipment should be limited to future road alignments."

9. "The value of mangroves as protection for the coast is considerable. The use of elevated structures, combined with mangrove forest preservation and management, make costly and environmentally damaging bulkheading unnecessary."

10. "Marina facilities should be designed so that tidal flushing will occur and so that the water will pass over a mangrove area before it reaches near-shore waters."

11. "Selective clearing in mangrove and marsh areas should follow these minimum guidelines to maintain water quality:

    a. "Basin forest should be left intact, with an adjacent buffer strip."
    b. "Riverine forest. Where there is some tidal flushing, the tall, red mangroves should be left intact, with a buffer as described for the basin forest. Channels may be cut through to increase tidal flushing. On the other hand, where there is no flushing, the tall red mangroves may be removed without affecting most marine life."
    c. "Fringe forest. Along the coast, and bordering canals and streams, fringing mangroves should be left intact in a buffer zone 50 ft inland of the high-water line. These trees may be trimmed to allow a view of the water from upland development. Mangroves seaward of the mean high-water line are protected by state law."
    d. "Dwarf forest. Where growth is very dense, areas bordering the coast and canals should be left intact, as in the fringe forest just described. Tidal channels may be cut through the forest and may actually increase productivity. Where growth is sparse and trees are so far apart that their branches do not touch, dwarf mangroves are not productive and do not contribute significantly to the environment.
    d. "Coastal marsh. Buffer zones similar to those recommended for the mangroves should be preserved in the coastal marsh. A buffer of marshland should be left intact adjacent to waterbodies and streams."

Source: Excerpts from Veri, A.R., et al, Environmental Quality by Design: South Florida. Coral Gables: University of Miami Press, 1975.

Table 4.7 - Guidelines and Standards for Coastal Projects. Elements Applicable to Residential Canal Systems.

Applicable Element	Specific Guidelines	Implementation
"Beachfront Protection and Management"	"Develop a shore protection program aimed at preserving the beach profile in its present slope and configuration."	1. "Structures should be used for beachfront protection only to supplement a nonstructural program. 2. "Sand for replenishing eroded beaches should be obtained from offshore deposits or from areas of active accretion." 3. "Inlet stabilization projects should be incorporated into the comprehensive shore protection program." 4. "Private and public projects to restore and stabilize dunes should be encouraged."
	"Implement regulations to protect the frontal dune system."	1. "Land-use regulations should be implemented to prohibit development in the frontal dune and beach area." 2. "Vehicle and foot traffic over the frontal dune system should be restricted." 3. "Beach and dune breeding habitats should be identified and protected during critical seasons."
"Bulkheads"	"Locate bulkheads shoreward of all wetlands, and design them for ecological compatibility."	1. "Natural methods of erosion protection, such as the planting of wetland vegetation, should be encouraged." 2. "Bulkheads or similar protective structures should be built above the annual flood mark in wetland areas." 3. "Bulkheads should be designed to be permeable to groundwater and runoff."
"Estuarine [and receiving water] flood protection"	"Combine and integrate estuarine [and receiving waterbody] flood plain management and ecosystem management programs."	1. "All structures on estuarine [or receiving waterbody] shores should be set back of the annual flood line." 2. "Elevate all structures placed in estuarine [or receiving waterbody] flood plain areas above the 100-year flood level." 3. "Areawide flood protection structures are to be avoided.
"Ground water extraction"	"Include controls on groundwater withdrawal in a comprehensive water management program."	Groundwater withdrawal should be controlled to prevent saltwater intrusion and land subsidence."
"Land Drainage"	"Maintain the quality, volume and rate of flow of coastal watershed drainage systems."	1. "Wetland areas should not be drained." 2. "Systems for the artificial drainage of shorelands should be designed to ensure that water leaves the project area in the quality, volume, and rate of flow prevailing in the natural drainage system."
"Marinas"	a. "Marinas and small-boat harbors should be planned to minimize the risk of water pollution."	1. "Marinas should be located on waterbodies where there is a high rate of flushing." 2. "Marina access channels should be designed to maximize circulation and avoid dead spots." 3. "Marina designs must incorporate facilities for the proper handling of sewage, refuse, and wastes."
	b. "Marinas and small-boat harbors should not alter the existing shoreline configuration or degrade vital habitat areas."	1. "Marinas should be planned so as to minimize the extent of excavation, shoreline alteration, and disturbance of vital habitat areas."

Table 4.7 - Continued.

Applicable Element	Specific Guidelines	Implementation
Mosquito Control	"Use appropriate water management techniques to control salt-marsh mosquitos."	1. "Open-marsh water management should be used as the primary means of salt-marsh mosquito control." 2. The use of impoundments to control salt-marsh mosquitos should be restricted to circumstances where open-marsh water management is not effective. 3. Restrict the use of pesticides to the application of short-duration compounds for urgent situations."
"Navigation dredging and spoil disposal"	a. "Design navigation dredging projects so as to avoid erosion, water pollution, circulation change, and disturbance of vital habitat areas."	1. "Navigation channels should be so located as to protect vital habitat areas and to prevent erosion of shorelines." 2. "Navigation channel dimensions should be kept to the minimum size" [consistent with circulation requirements for the system]. 3. "Dredging operations should be suspended during critical periods of fish migration and breeding. 4. "Dredge types should be selected that will minimize operational environmental disturbances."
	b. "Control dredge spoil disposal to protect vital habitats and estuarine [or receiving water] quality."	1. "Alternative methods should be used to avoid disposal of spoil in open estuarine [or receiving] waters or on vital areas." 2. "Handling, dewatering, and disposal of spoil should be controlled so as to prevent water pollution."
"Piers and Docks"	"Limit the encroachment of recreational boat landing facilities into wetlands and coastal waters."	1. "Piers should be built on pilings rather than solid fill." 2. "Proliferation of individual piers should be discouraged in favor of mooring buoys and shared community landings" [although individual piers may be advantageous, in some instances, in particular locations, depending on the layout and design of the residential canal system].
"Residential Development"	a. "Review all residential development and construction applications in shorelands for compliance with ecosystem protection requirements."	1. "Vital areas should be exempt from residential development." 2. "Residential development should be fully controlled to prevent degradation of the quality, volume, and rate of flow of the natural drainage system of the watershed. Specific factors to be controlled are: runoff water, drainage of wetlands, erosion, location and total area of impervious surfaces, storm sewers, groundwater withdrawal and recharge, sewage and septic tanks, solid waste disposal, and residential land care."
	b. "Establish a special review process for waterfront and floodplain residential development and construction proposals."	1. "Residential development proposed for floodprone areas must be specially controlled and must comply with federal flood insurance criteria." 2. "Special constraints should be imposed on waterfront development to preserve shoreline configuration, protect vital areas, and avoid water pollution. Special problems include: beach protection, dredging, boat facilities, setbacks, and roadways." 3. "Excavation and fill to create canalside waterfront homesites should be prohibited" [unless such development can be shown to have the potential of improving the existing productivity and aesthetic qualities of the site without destroying wetlands].

Table 4.7 - Continued.

Applicable Element	Specific Guidelines	Implementation
"Roadways and Bridges"	a. "Locate road systems to avoid impingement on vital habitat areas or interference with surface-water or ground water flow."	1. "Roadway systems should be located so as to avoid vital habitat areas." 2. "Grade-level roadways across the lower floodplain should be located parallel to the path of water flow."
	b. Elevate roadways over water areas and otherwise design them to avoid alteration of vital habitat areas and to minimize disruption of water flows in water areas and floodplains.	1. "Causeway design should elevate the roadway on pier or piling supports and avoid the need for solid fill."
	c. "Utilize construction and maintenance methods that do not alter wetlands, intertidal marshes, or other vital areas and that do not adversely affect water quality or water flow."	1. Construction operations should adhere to all requirements for water quality and general ecological protection of coastal waters." 2. Schedule construction phases to avoid critical periods of breeding, feeding, and migration of coastal species."
"Septic Tank Location"	"Locate and maintain septic tank systems so as to avoid water pollution"[provided it has been objectively determined that septic tanks are preferable over centralized sewage treatment].	1. "The disposal or absorption field of a septic tank system should be set back at least 150 ft from the annual high-water line"[or a greater distance, depending on soil characteristics]. 2. "Septic tank systems should be installed only when the highest annual groundwater level is at least four ft below the absorption field." 3. "Septic tank systems should be installed only when soil characteristics are suitable."
"Sewage Treatment Systems"	a. "Locate sewage treatment plants, outfalls, pipe systems, and storm sewers so that they do not disrupt vital habitat areas."	"Select routes for collector systems and sites for sewage treatment facilities that avoid vital habitat areas."
	b. "Take immediate action to upgrade municipal sewage treatment and disposal systems for federal effluent standards."	1. "Land application of treated sewage effluent should be used whenever practicable." 2. "Tertiary treatment should be required for sewage effluent to be discharged into typical estuarine [receiving] waters." 3. "Ocean [or offsite] outfalls should be located at the greatest practicable distance from shore and designed to provide maximum dispersal of the effluent." 4. Stormwater and industrial sewage collection and treatment systems should be separated from sanitary sewage systems to the maximum extent possible." 5. "Sewage sludge should be recycled through land application where practicable."
"Solid Waste Disposal"	"Locate solid wate disposal areas so as to prevent the pollution of coastal waters.	1. "Vital habitat areas should not be used as landfill sites." 2. Sanitary landfills should be located in areas of suitable water characteristics and soil permeability."

Table 4.7 - Continued.

Applicable Element	Specific Guidelines	Implementation
"Tract and Site Preparation"	a. "Exercise strict controls on erosion during site preparation and construction."	1. "Buffer strips of natural vegetation and artificial detention systems should be used to control erosion." 2. "Construct runoff diversions in conjunction with sediment removal methods to minimize the extent to which exposed soils are eroded." 3. "Ground surfaces should be stabilized immediately after any action that destroys the natural vegetational cover and leaves soils exposed to erosion forces."
	b. "Preserve wetlands and other components of the natural drainage system of the area under development."	1. "Drainage of high-water-table areas should be discouraged." 2. "Wetlands should be allocated for purposes that require only light-duty structures."
"Urban Runoff"	a. "Limit impervious surfacing to the minimum possible."	"Development should be planned to utilize permeable surfaces wherever feasible."
	b. "Design storm drainage projects to simulate the natural pattern as nearly as possible and to reduce the need for storm sewer systems."	1. "Stormwater runoff should be diverted and dispersed into areas of natural vegetation and soils wherever feasible." 2. "Delay stormwater runoff with detention systems for dispersal and release at a flow simulating the predevelopment state."
	c. "Plan for separate collection of storm runoff for utilization where possible and for separate treatment where not."	1. "The stormwater collection and treatment system should be separated from the sanitary system whenever feasible." 2. "Specific concentrated sources of runoff contamination should be located and isolated for special treatment."

Source: Clark, J.R., 1977, pp. 287-556.
Note: Words in brackets added for canal systems.

Table 4.8 - Design Guidelines Relative to Canal Design by Planners and State Agencies.

EROSION...

Not Considered	Considered
"Soil is extremely vulnerable to erosion when vegetation is carelessly removed. What little topsoil exists may be carried away. The resulting sedimentation will destroy fish and other aquatic life and can cause unwanted growth in waterbodies.	"By leaving vegetation undisturbed, root systems stabilize the soil and help prevent erosion while providing the necessary microclimates for wildlife. Where clearing is necessary, subclimax pine should be removed before hardwoods.
"Vegetation provides a sense of enclosure. Removal of vegetation can result in increased sun, salt spray, and wind damage.	"Vegetation complements the topography. Careful pruning will open views without destroying the positive effects of climatic control and wind protection. Enclosure is maintained through careful removal of vegetation adjacent to a housing site.
"The character of the site and the privacy which it affords can be lost through the removal of vegetation."	"The natural character of the coast is retained when vegetation is emphasized as a site feature. This character can be a strong selling point when marketing the product.
"Highly permeable soils permit sewage to pass through too rapidly for purification. Often, the result can be the degradation of groundwater and nearby wells. Test borings must be conducted prior to construction.	"In suitable soils, sewage passes at the proper rate to permit chemical reactions to occur and bacteria in the soil to naturally purify. Chances of groundwater contamination are minimized.
"Building on unsuitable soils may result in foundation failure, settling, and damage to pipes and walls. Although this may be overcome with technology, the expense may be prohibitive.	"Soil must be able to support the weight of the structures. Shifting and settling are minimal on suitable soils. Have soils analyzed by soils or geologic engineers prior to construction.
"Failure to analyze the ability of the soil to support different types of vegetation can result in death of landscape materials."	"Soils and vegetation are closely related. Some soils will require much improvement before plants will grow. Check to see if topsoil is available and worth saving prior to development, especially in areas where vegetation exists. Certain soils have a toxic affect on some species of plants."

NATURAL DRAINAGE...

Not Considered	Considered
"Failure to observe natural drainage patterns can lead to flooding, improper septic tank functioning, and damage to structures.	"Observing natural drainage patterns and defining where flooding may occur will lead to a safe location for development.

Table 4.8 - Continued.

Not Considered	Considered
"Failure to control runoff during site preparation will result in erosion. This condition causes siltation of waterbodies plus a loss of vegetation. Grade only the minimal amount of land required for construction.	"Controlling runoff through the use of sediment basins, careful grading techniques, and prompt seeding of graded areas will minimize erosion and siltation of adjacent waterways.
"Failure to protect areas from compaction and destruction by heavy machinery, will result in damage to vegetation. When this trouble has occurred, it is financially expensive to correct.	"Carefully defining those areas requiring protection with fences, tree guards, bollards, and signs will minimize destruction of crucial vegetation. Specifications must provide for this type of protection.
"Failure to consider stockpiling techniques and site design specifications may result in natural resource damage to the site, as well as a loss of time."	"Proper stockpiling techniques often minimize the area required while protecting the surrounding from abuse. Site quality control and construction should be carefully supervised by a professional. Site inspection at regular intervals is necessary to assure that the project is being carefully executed."

SITE

Not Considered	Considered
"Poor construction supervision and failure to protect vulnerable pipelines can result in destruction of underground infrastructure.	"Marking septic lines with flags will help prevent their destruction or alteration by heavy equipment during construction.
"Grading adjacent to buildings with heavy equipment can result in damage to water-proofing, positive drainage and the structure itself.	"Final grading when carried out by hand or very light equipment will prevent damage to buildings, foundations, and vegetation.
Failure to check the compatibility of pH and nutrient content when importing topsoil from other areas may result in death to existing vegetation. Runoff from such soils may also negatively affect nearby vegetation.	"Specifying imported topsoil of the same pH as that on the site will help to insure that existing vegetation is maintained.
It can become very costly to replace topsoil that has not been properly retained during construction.	"Topsoil is a valuable resource to the builder. He should provide for stockpile areas which will not be contaminated by foreign construction matter or affected by wind.
Try to maintain existing vegetation and do a minimum of tree thinning. The use of heavy equipment often causes compaction and change of grade resulting in a loss of vegetation. Stockpiling under trees may also affect growth.	"Grubbing or thinning of a site is best accomplished with small equipment and should be prohibited within the foliage lines of trees. Marking trees with plastic survey tape will insure positive identification and easy removal. Damaged limbs should be cut and treated with healing compound.
Chopping of tree roots to facilitate underground pipes can easily curtail growth of trees.	"Retaining roots over approximately 75mm in diameter when digging trenches will usually insure that vegetation will live. Pruning of trees will help to offset any root loss.

Table 4.8 - Continued.

NATURAL DRAINAGE (cont'd)	
Not Considered	Considered
"Failure to consider natural drainage through careless grading and indiscriminate use of culverts results in erosion problems, loss of existing vegetation and damage to wildlife habitats.	"Small streams are too often ignored as a potential landscape feature. Minimal treatment of stream channels with the use of bridges or well designed retaining walls can lead to highly desirable landscape amenities.
"Destruction of natural channels and streambank vegetation also destroys the natural beauty and diversity of the environment."	"Several coastal vegetation types are highly sensitive to changes in water level. Check to see if such species exist on your building site."
SETBACK...	
Not Considered	Considered
"Inadequate setbacks near the marsh [or canal] edge increase the chance of water pollution from septic tanks and saltwater intrusion in wells.	"Setbacks insure septic system runoff will be adequately purified by passage through the soil to help prevent pollution of water. Problems resulting from saltwater entering shallow wells are also reduced.
"Without a setback, the building is exposed to strong winds, possible wave damage, and glare from sunlight. Flood damage risk is also increased.	"Retaining existing salt tolerant vegetation along the edge of saltwater bodies helps insure the health of inland vegetation and protects development from storm damage.
"Buildings become highly visible from the water when placed immediately on the edge of the water or marsh. This detracts from the natural character of the site."	"By using vegetation as a screen, the natural character of the site is retained for dwellers and boaters. Careful pruning of the shoreland vegetation opens an interesting view to the water while maintaining privacy."
SURROUNDING FEATURES...	
Not Considered	Considered
"A direct loss of privacy for both the dweller and park visitor [can occur] due to a lack of proper site planning.	"Advance planning for parks and access to water can increase the enjoyment of both visitors and residents.
"A lack of planning [can result] in a hodgepodge of wires, gaudy signs, and undesirable views from houses.	"Utilize underground utilities, service roads, coordinated signs and vegetation to enhance community appearance and property values. Where underground wires are not feasible, utility poles can be selectively placed to reduce visual impact.
"Poor land use planning destroys the quality and character of potentially high-value areas."	"Retain the natural character by planning with the land."

Table 4.8 - Continued.

[illegible]	
Not considered	Considered
Sewage lines adjacent to trees which are not enclosed by concrete can be clogged by root growth	When roots are considered to negatively affect sewage lines, the lines can be encased in concrete.
Aligning roads and walkways close to trees can result in vegetation loss and/or pavement damage.	Align roads to meander between larger vegetation (but not too close).
"Solid or slab type foundations do not allow the tree's roots to 'breathe'. When siting a building, existing vegetation and root structures should be considered.	Foundations allowing an air space between roots and structure will allow roots to live.
"Removing soil from around trees can easily result in death if fibrous root damage occurs.	When adding soil, either add it at a maximum of 4 inches per year or utilize a type of tree well.
'Adding relatively small amounts of soil to the existing grade can result in vegetation death.	"Never remove soil from inside the foilage line of trees."

Source: Excerpts from: Resource Planning Section, Handbook: Building the Coastal Environment. Office of Planning and Research, Georgia Department of Natural Resources, Atlanta, Ga., 1975, pp. 78-109.

Table 4.9 - The Rational Approach to Canal Design, Snyder Oceanography Services

GOAL	REQUIREMENTS	ENGINEERING CRITERIA	RESULTING DEVELOPMENT	ENVIRONMENTAL IMPACT
	OFFSITE SEWAGE TREATMENT OR PACKAGE PLANT	ELIMINATE DOMESTIC WASTE SEEPAGE		ELIMINATION OF POLLUTANTS
		ELIMINATE DIRECT STORMWATER DISCHARGE	BALANCE SALT-WATER INTRUSION WITH FRESHWATER HEAD	NATURAL WATER 'TREATMENT'
	OFFSITE WATER SUPPLY	RECHARGE GROUND-WATER ON SITE		GROUNDWATER RECHARGE
RESIDENTIAL UTILIZATION OF WATERFRONT	RETENTION OF SOME SHALLOWS	NO DRAWDOWN OF WATER TABLE	VEGETATION FOR NUTRIENT UPTAKE	
		DESIGN SHALLOW AREAS FOR REVEGETATION		COUNTERACTING SALTWATER INTRUSION
WITHOUT ENVIRONMENTAL DEGRADATION	HABITAT DIVERSITY	USE COMBINATION OF STABLE SLOPES, RIP-RAP & VERTICAL BULKHEADS	VARIED HABITAT FOR EURYHALINE SPECIES & NURSERY GROUNDS FOR NERITIC SPECIES	GOOD MIXING
	INCREASE CIRCULATION	INCREASE UPLAND WATER AREA	'NATURAL' LOOKING WATERWAYS	
		FOLLOW NATURAL DRAINAGE SLOUGHS	NATURAL PRESERVES FOR BIRDS AND SHORE CRITTERS	REDUCED DEBRIS COLLECTION
		ELIMINATE NARROW DEAD ENDS		REDUCED SHOALING & EROSION
		'NATURAL' LAYOUT WITH GENTLE BENDS		BETTER FLUSHING
	PROMOTE MIXING	BALANCE DEPTHS WITH WIDTHS & SHALLOWS		AMENITIES OF NATURAL LAND AND WATER SCAPING
		PROVIDE ROUGHNESS (BANKS & BOTTOMS)		
	ELIMINATE SCOUR FOR SPECIFIED CONDITIONS	PROVIDE STABLE SECTIONS		

SNYDER OCEANOGRAPHY SERVICES

THE RATIONAL APPROACH TO CANAL DESIGN

CHAPTER 5

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

The design of a canal system for a particular site necessarily begins with an evaluation of the site characteristics. This evaluation establishes the overall setting in terms of the features and conditions which can and cannot be altered, and determines the broad limits within which the design will have to operate. 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, which are summarized in Table 5.1, include its area and boundaries, topography and drainage, tidal range, climate, hydrology and water resources, pollution sources and water quality, natural features (soils, vegetation, wildlife), existing residential offsite and onsite communities, special

areas (for preservation), and aesthetic features. Some of these characteristics may be considered to be fixed, such as the area and boundaries, general topography, climate, tidal range, certain aspects of the water budget, some pollution sources, soils, principal ecosystems, some of the aquatic life, existing communities, special areas and neighboring sites. The canal designer must design around these characteristics in such a way that the new community has a net positive impact on the site and results in an overall improvement in the environment. Other characteristics are somewhat alterable, such as topographic details, drainage, some components of the water resources, possibly some pollution sources, and the vegetation.

For any canal site development certain characteristics will have to be mapped. Many kinds of maps and charts showing particular features of land and water areas are available, but these almost always cover an area far larger than the area of interest, without the degree of detail required for design. Aerial photography may be used for obtaining some information, but it will also be necessary for the owner and the designer to walk over the area and observe the condition of the important characteristics. It will also in many cases be necessary, and in all cases advisable, that a site survey team consisting of professionals in each of the major areas of interest be assigned the job of quantifying those characteristics.

This chapter is concerned with the problem of measuring or otherwise quantifying the relevant characterstics of a site. It will cover sources of existing information which may be applicable, as well as the more difficult task of measuring those features which either have never been quantified before, or were quantified too long before to still be reliable.

5.1 Fixed Characteristics

5.1.1 Topography and Drainage

The overall topography of a site, which includes its shoreline as well as the large-scale rises and depressions which would be too expensive to modify, 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 often 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.

U. S. Geological Survey (USGS) topographic maps are useful for large area coverage. If a site has not been selected, and a large area is being surveyed for a suitable location, a combination of topographic maps, aerial photographs, and coastal charts will be useful in searching for suitable shorelines. Once a site has been selected, however, USGS topographic maps will not provide the detail necessary for design. Aerial photographs are then appropriate, and are the least expensive way to obtain a map of the existing shoreline, natural features, and existing channels. Such photography is limited, however, in not being able to penetrate thick vegetation, which is often especially heavy near the shoreline. Stereo pairs with suitable ground control are a useful method for obtaining topographic information. The interpretation of stereo aerial photography, however, requires the services of a professional with the proper optical analysis equipment. The minimum contour interval which can be accurately obtained from aerial photographs depends upon

the altitude from which the photographs are taken; the smallest contour interval available is approximately 1 ft for each 1,000 ft altitude [Soil Conservation Service, 1969, p. 1-75]. A scale of 1 inch to 200 ft (1:2,400) with 5 ft contours is generally satisfactory for preliminary work and is usually available at a reasonable cost. Such a map can be enlarged photographically if desired.

Applicable maps and aerial photographs are often available from the National Cartographic Information Center (U. S. Geological Survey), local or county government agencies, planning groups, and local libraries and archives. They are also available from commercial sources. Some specific additional sources are listed in Morris, Walton and Christensen [1978, Section 12.3]. A chronological series of such pictures of the area around the site will be helpful in assessing past conditions which may have affected the characteristics of the site. A search for such information may uncover other sources of information which would otherwise not have been discovered.

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

1. property boundaries
2. vertical contour intervals of 5 ft.
3. scale, date, source of topography, north arrow
4. major physical features such as existing canals, streams, unusual trees or vegetation, paths or roads, depressions, etc.
5. extent and type of vegetation

6. easements, rights-of-way, adjacent roads

7. locations of nearest available utility taps

One convenient method for analyzing the topography is to subdivide the topographic map into areas of various incremental slopes, e.g., areas with 0 to 1 percent slopes, 1 to 5 percent slopes, etc. This will assist in vizualizing the topography for a trial layout and a preliminary site grading plan.

5.1.2. 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* [NOS, (annual)]. This method should be used with caution, however, since the tide can be either attenuated, amplified, or shifted in phase. This can occur in both shallow coastal waters and areas with complex shorelines, and is often caused 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. A tidal boundary is an imaginary line of intersection between a horizontal plane representing a mean water level and the sloping, irregular shoreline. Since the slope of the near shoreline along the Atlantic and Gulf Coasts is on the order of 1:500 or less, for every elevation difference of 1 inch there will be a boundary displacement of about 50 ft. Thus, the matter of accurately defining a mean water line can be very important from a legal point of view.

The basic difficulty in establishing a mean water level lies in the harmonic complexity of the tide and in the constantly changing level of the shoreline through erosion and accretion. The approach which seems to be generally acceptable in courts is to obtain no less than one year of local tidal records at the site and to tie those records into the nearest location with at least 18.6 years of record. However, the courts have also used title histories, vegetation lines, and other criteria in resolving these issues [Bockrath and Polis, in Clark, 1977, pp. 738-740]. Thus, 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 Morris, Walton and Christensen [1978, Section 12.3].

5.1.3 Climate

Data on wind, precipitation, and other climatic variables are available for selected areas in all states. Figure 5.1 shows the location of weather service offices and other stations in Florida, the legend identifying the kinds of data reported from each station. Daily precipitation and three-hourly wind data are available, for each of the stations identified by a double circle in Figure 5.1, from the National Climatic Center.

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 Morris, Walton,

and Christensen [1978, Section 12.3]). 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. To test the possibility of a correlation, for verifying the three-dimensional numerical model during this study, the half-hourly measured winds at the 57 Acres canal site during the period of October 17 through 21, 1977, were compared with the (interpolated) one-hour wind data for the same period at West Palm Beach airport. Two separate linear correlation coefficients were calculated, one for the wind speeds and one for the wind directions. These two correlations were made for all of the wind speeds in the data set, but not for all of the wind directions, since a wind direction associated with a wind speed of zero is indeterminate. The correlation coefficient for wind speed was 0.73, and for wind direction was 0.28. It was therefore concluded, from this one comparison, that it is unlikely that winds recorded offsite will be very representative of local wind conditions.

For the simulation of the performance of a proposed canal network, however, the use of historical wind records is adequate. The proposed or trial canal designs should be tested for typical conditions during two or more seasons, and for these tests a representative set of variables should be established. 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 Morris, Walton, and Christensen [1978, Section 12.3].

5.1.4 Hydrology and Water Resources

The canal designer must make decisions that affect the supply, movement and quality of surface and ground water 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 surface water, and locating and quantifying groundwater resources.

General information on a county-wide basis regarding the geologic and hydrologic background of the area, availability of data, rainfall and evapotranspiration, the surface water and groundwater systems, water quality and water use may be found in Reports of Investigations published by the Florida Bureau of Geology. Maps of certain specific hydrological and water resources features, state-wide and regional, are also published by the State Bureau of Geology. Streamflow, surface water quality and groundwater quality are monitored and recorded on a daily basis and published in the annual summaries, organized by state regions, by the Office of Water Data Coordination (OWDC), USGS. These reports and maps, in addition to whatever information can be obtained from local offices of federal, state, regional, county or municipality regulatory or planning agencies, may provide part of a data base from which a site study can begin.

In a preliminary site investigation the canal designer should primarily act as an observer. He should look for existing bodies of surface water, and if he finds any, ask himself whether they might lie

in a flood plain. He should 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 see if there are any obvious drainage paths into or out of the waterbody. He should also take a surface and bottom sample, checking its pH, chloride content and DO. He should consider the scenic quality of the site, 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 presence 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. If there are, special provisions may have to be incorporated into the canal

design. The other important boundary is the shoreline and the tidal entrances, or planned entrance locations, on the receiving waterbody. This shoreline, and the entrances, also provide information on physical trends and the quality of the water which may be of importance in future design decisions.

Effects on groundwater are more difficult to assess. First, the canal designer should know how the planned community should have to 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 prinicpal 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 remote supplies, a hydrologic survey by a competent 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 for information on water data, water supply and waste water disposal are listed in Morris, Walton, and Christensen [1978, Section 12.3].

5.1.5 Vegetation

One of the most important features of a site is its pattern of natural vegetation. In addition to the significant role vegetation

plays in stabilizing soils, providing food and protection for wildlife, photosynthesis, and providing scenic quality, it also 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 (nonindigenous) species.

The canal designer, with the services of a competent professional advisor, should include a survey and mapping of the types of vegetation at the site in this 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 the particular area may be in a state of establishment or erosion. If it is in the process of establishing itself naturally, this

is an indication that soil and water conditions are 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 its water quality.

Some sources of information on vegetation are listed in Morris, Walton, and Christensen [1978, Section 12.3].

5.1.6 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 drain 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 they may 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. Dept. of Agriculture 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.

Soils are arranged in layers *(horizons)* which can differ substantially from one to another. A *profile* is the sequence of horizons extending downward form the surface. If canals have been dug, but not improved, on a particular site, this provides a unique view of the existing profiles. Soils that have very similar profiles are called *soil series*, which are used by the SCS to classify soils. The SCS publishes soil surveys for each county which, besides describing in detail the characteristics of each soil that has been identified, contains a general soils map which delineates the soils by *mapping units* or by *associations* . Mapping units are subgroups of a soil series which describe the particular characteristics that vary within a soil series, and are located on a soils map in three to ten acre units. A soil association is an area or landscape that has a distinctive proportional pattern of soils, usually mapped in fifty to one-hundred acre minimum units. SCS personnel can provide assistance in interpreting soils maps and in identifying the soils within a mapping unit that cannot be resolved by aerial photography.

The canal designer should try to delineate those areas which are suitable for construction and the movement of heavy equipment, and those areas which have soils that cannot withstand compaction. He should look for areas which drain well naturally and would be suitable for detention ponds, and obtain information on soil profiles which will help to locate areas which are suitable for septic tanks.

Some sources of additional information on soils are listed in Morris, Walton, and Christensen [1978, Section 12.3].

5.2 Alterable Characteristics

5.2.1 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 site 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. A prepared checklist of questions

similar to the following, for identifying soil erosion problems, would be useful during the site survey [N.H.B.A., 1972, p. 251].

1. Nature and type of soils: are they erodible?
2. Steepness of topography.
3. Vegetation: wooded? grassed? other?
4. Present degree of erosion: natural swales? ravines? condition of soils and vegetation in ravines?
5. Amount of grading that may be required.
6. Offsite (downstream) questions:
   a) Clearly developed drainageways?
   b) Drainageways obstructed?
   c) Potential damage to downstream property?
7. Offsite (upstream) questions:
   a) Other development taking place?
   b) Rapid discharge of upstream water taking place in watershed?

Some sources of information on drainage are listed in Morris, Walton and Christensen [1978, Section 12.3].

5.2.2 Pollution Sources

Potential sources of pollution, which have been characterized in Section 2.5.2, 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 of pollution 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. If the canal designer is able to locate significant offsite sources of pollution which have the potential of adversely affecting his project, it should be possible to have that source stopped or diverted through appropriate regulatory action.

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 that air quality standards cannot be met.

Pollution sources originating on the proposed site must also not be able 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.

5.3 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. This section will describe the objectives of the preliminary site investigation and the subsequent field surveys.

5.3.1 Objectives of Field Work

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 investigation first, followed by one or more field surveys to obtain data for design. The objectives of the preliminary site investigations are:

1. To determine the conditions that 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.
2. To determine if any unusual conditions are present that will require additional measurements, special equipment, or special procedures.
3. To measure depth in canals for later determination of support equipment requirements.
4. To determine the approximate range for the variables to be measured (which determines the type of instrumentation and support equipment that will be required).
5. To locate elevation reference points for future surveying.
6. To familiarize field crews with site logistics.

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

1. Support the design analysis for the preparation of the trial canal system designs.
2. Enable estimates to be made of coefficients for verification of the 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 at the site, 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.

5.3.2 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, 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 dye concentration resulting from a planned dye release until the dye becomes undetectable. 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 field personnel 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 times 3 surface locations, times 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 these measurements at all thirty-three locations. Such an approach certainly would not provide useful data, as there is too great a time span involved.

Another example will demonstrate the practical limits to sampling with small field crews. Given that a one-hour time period is reasonable for obtaining an instantaneous picture of these four parameters in the

canal, that two boats are available and 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 be over an hour and a half. These measurements, furthermore, could be made and recorded by hand, at a rate of approximately one per minute, or could be sampled and recorded at a faster rate with a completely automatic instrumentation system.

It may be concluded that a field survey requires precise planning, reliable equipment, good training and considerable endurance if usable simultaneous 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 that has had experience in coastal oceanographic and canal hydraulic measurements. The canal designer should be satisfied that this organization has the proper instrumentation and equipment, and available experienced field crews, 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 performance specification defining in detail 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.

In the event the canal designer decides to direct the field surveys himself, one alternative to the problem of obtaining the necessary instrumentation is rental. For short-term surveys, rental equipment is less expensive since leasing companies maintain the equipment and it can

be supplied propmptly when and where needed, with no capital investment. Unfortunately, specialized instruments such as electromagnetic current meters and fluorometers are not ordinarily available from leasing companies.

5.3.3 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 5.2 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 the canal system is 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 inch to 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.

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, or at the location of the maximum current if this can be determined to be off the geometric centerline. 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, however, that such uniform conditions do not prevail very often in Floridian canals.

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 high 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 flow into the canal system could exist should be evaluated to determine their possible effect on the structure of the density circulation. Standard techniques for the generation of storm hydrographs (such as the methods provided by the Soil Conservation Service) can be used for a given site provided soil maps are available. The drainage pattern of the land on which any canal system is to be constructed must be designed into the system, and may require field elevation data taken by surveying to supplement existing topographic data. Rainfall measurements are useful in calibrating calculated storm hydrographs, particularly if a significant source of runoff is located on the site.

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. In particular, the State of Florida has established five water usage classifications, and in general requires that no modifications be made in the vicinity of any water falling under state jursidiction that will result in lowering the existing classification.

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 DO. However, a system which has good flushing characteristics throughout, and which is designed to control pollutant inputs by adequate drainage and waste water handling, can be expected to meet these criteria.

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 and should indicate, after the introduction of a quantity of dye at an appropriate location, the rate of spreading and the flushing time of the waterbody into which it has been introduced. Variations in water quality in a

given canal system, for a given pollutant loading distribution, can be predicted with the numerical canal model (CANNET3D) as described in later chapters. Diffusion and dispersion coefficients for the model can be inferred from the results of dye studies.

Bed and bank stability can be determined, for a given canal geometry, from soil samples. Analysis of the soil samples provides the critical shear stress above which erosion will occur, and below which deposition will occur. Water velocity measurements or predictions can be analyzed to determine the order of magnitude of bank and bottom shear stresses, which then permit an estimate of the probablity of erosion or deposition.

Table 5.1 - 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

Table 5.2 - Relationship Between Desired Information, Information Derived by Engineering Analysis, and Variables Measured During Field Surveys.

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
	$\tau_{critical}$	

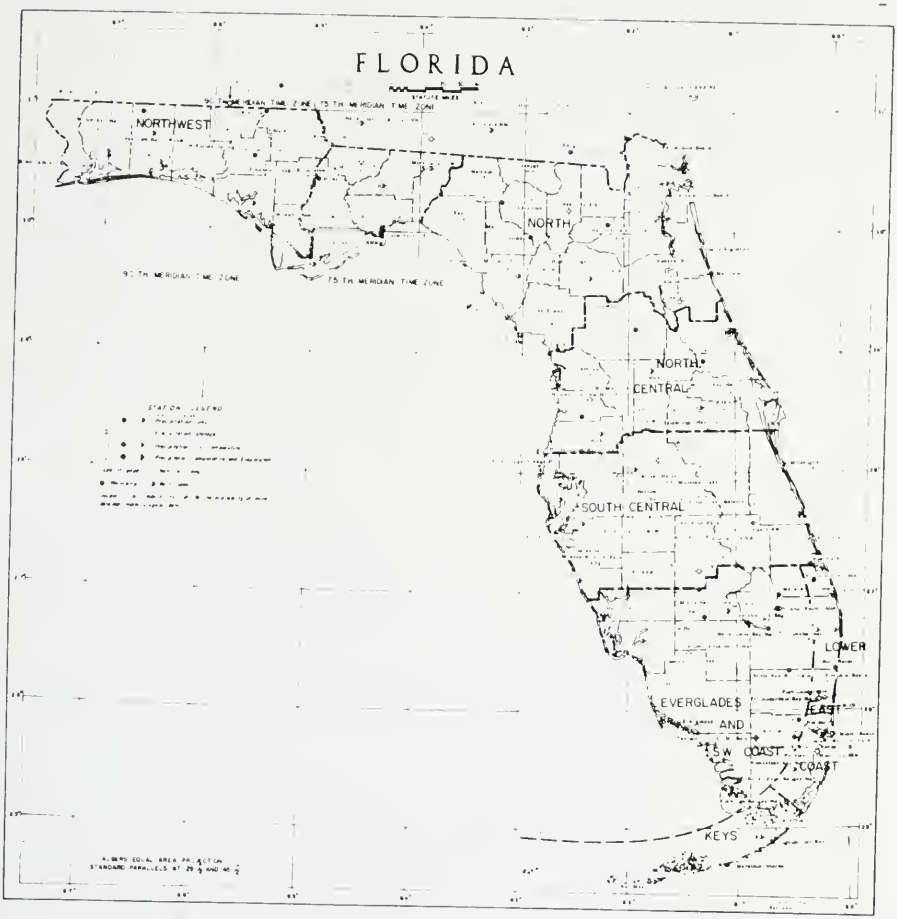


Figure 5.1 - Location Map of Weather and Meteorological Stations in Florida

CHAPTER 6

FIELD MEASUREMENTS, INSTRUMENTATION, AND RESULTS

As stated in Chapter 1, the field measurements portion of the project had several different objectives. The first, chronologically, was to determine the significant features of tidal canal hydrodynamics and mass transport for purposes of developing the numerical model. This work began in 1975, but was not completed until 1977 when the final comprehensive field surveys were made in the 57 Acres and Loxahatchee River canals. The concepts resulting from this work have been presented in Chapter 2, where the features of Floridian canals were described. The field work supporting those concepts will be summarized in this chapter.

The second principal objective of the field work, to develop effective measurement techniques and to determine instrumentation requirements, had been essentially completed by the end of the second year. Refinements were added during the third year, but the essential equipment had almost all been acquired by the time of the comprehensive field studies during 1977. The instrumentation, measurement procedures, and results of the field work are summarized in this chapter.

The third objective was to obtain the data necessary for verification of the numerical model. This was accomplished during the comprehensive field surveys in 1977, when velocity profiles, salinity profiles, wind and dye profiles were sampled simultaneously over an entire reach.

This chapter spans the portion of the canal design process that begins with the quantitative evaluation of a site or an existing canal network and ends with a trial canal network design. Following discussions of desirable specifications for canal instrumentation, the equipment used for the study by the Hydraulic Laboratory is described. Then and the reduction and presentation of data, and the results of three years of field observations in Floridian canals, are summarized in terms of their contribution to the canal design procedure.

6.1 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,

a) Accuracy: 1 to 5 percent full-scale accuracy 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.

b) Reliability: Best available reliability is required. The high cost of a field survey demands that all equipment be operational for the entire period of the survey.

c) Power source: Operation from 12 volts DC is preferable, provided either that built-in rechargeable cells with low power consumption are supplied, or sufficient regulation and protection are provided for direct connection to lead-acid batteries. The latter is preferable

provided the boat has an alternator for recharging the batteries, and provided the boat will be run often enough and at a high enough speed to maintain battery charge. 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.

d) Environmental conditions: Salt moisture, direct sunlight, rain squalls and relative humidity approaching 100 percent at night are some of the environmental conditions that must be planned for. Equipment with mechanical parts requires 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.

As an illustration of the importance of environmental conditions, the Hydraulic Laboratory lost an entire set of ten meters which display current velocity when an instrument box was left in the sun for several hours. The box was hermetically sealed to protect the electronics, but the cover was glass and the heat that built up inside the case warped the plastic meter cases. In another instance, when around the-clock dye concentration measurements were being conducted, it was found that a great deal of moisture condensed on all surfaces at night, and all equipment not designed for protection against moisture had to be kept covered in waterproof boxes except during actual operation.

e) Simplicity of operation: For many reasons, simplicity of operation is one of the most important attributes 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 so busy that they must delegate the operation of equipment to

others. 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.

f) 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 should be desiged to serve as a check on whether the equipment is functioning properly.

g) 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. Since automatic locating and recording systems for boat position are not usually available for laboratory survey craft, manual recording of navigational position on a strip chart is often the only practical solution.

6.2 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 so that the reliability of the data may be assessed by other investigators familiar with the equipment, and to serve as a guideline for those planning to do similar work. Table 6.1 summarizes the types of equipment and instrumentation that can be used for each of the measured variables listed in Table 5.2.

6.2.1 Tide Recording

A reliable tide recorder is essential for canal survey work. A number of different water level recorders are available and are 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 electrical power, but it is still necessary to wind the clock and replace the recording media at regular intervals. The Hydraulic Laboratory has used the Leupold Stevens Type F, Model 68, and found it 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 damping 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 to a great extent on the length of time the gauge will be installed. The structural requirements for the particular arrangement to be used should be determined for the length of time the gauge is to be installed, to withstand expected climatic conditions and natural and man-caused interference, and to provide security for the equipment.

The Hydraulic Laboratory's recording tide gauge is mounted on a stilling well in a box that can be locked, as illustrated in Figure 6.12.1. The float is kept separate from the counterweight by providing a separate PVC pipe parallel with and at some distance from the float pipe for the counterweight, to minimize interference with the float cable. The stilling well can be either mounted on a platform, such as a dock or

bulkhead, or supported free-standing along the side of the canal. If the bottom of the stilling well is suspended in the water, it is necessary to block the end with a cap to dampen wake effects. All but one of a series of holes in the side of the stilling well are taped with duct tape before installing the stilling well, which ensures maximum damping.

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 that 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. The geodetic net is referenced to sea level datum (SLD), which is the USGS 1929 datum plane, (also called the national geodetic vertical datum, NGVD). SLD is referenced to MSL at 135 of the 272 tidal benchmarks that have been established in Florida. The measured elevation referenced to SLD may be converted to an elevation referenced to MSL in accordance with procedures described by Piccolo [1976]. All tide records taken by the Hydraulic Laboratory for the

canal design research project were leveled to the nearest benchmark and referenced to MSL.

6.2.2 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 routine and well within accuracy requirements. The fiberglass open-reel tape is preferable over steel tape because it is unaffected by saltwater and sand.

All critical distances measured by the Hydraulic Laboratory have been taken with a surveyor's tape or, in the case of the 57-Acres canal network, from an aerial photograph. In some of the early dye dispersion studies the distance from the injection point to the sampling point was taken with an optical rangefinder, but this technique was later abandoned and all critical distances were taped.

6.2.3 Depth Recording

A Benmar echo sounding recorder, Model DR-68, is used for taking depth profiles. This instrument is mounted in the survey boat, and the transducer is secured to the transom so that the head is a few inches below the surface of water. The instrument is powered by 110 VAC, which requires that an inverter be provided to convert the 12 VDC boat supply. The recorder 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. It is useful to place stakes at 100 ft intervals along the side of the canal for assurance of accuracy in relocation of stations.

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

All longitudinal depth contours measured by the Hydraulic Laboratory were recorded with the Benmar depth reorder, except transverse depth profiles which were taken by tying a half-inch line across the canal and using a graduated pole to measure the depth at ten foot increments.

6.2.4 Current Measurments

At the beginning of the canal project an Ott laboratory propeller meter, Model C.1, was used to measure current speed. This instrument, while very useful for measurements in a laboratory flume, was found to have 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 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, the

propeller meter is a pulse counting system which requires that the reading be integrated over at least a minute, and often over several minutes if significant differences in successive pulse counts are obtained.

After experiencing difficulties measuring very low currents, a Savonius-rotor current meter was borrowed from Snyder Oceanography Services, Inc. of Jupiter, Florida. This instrument, which had been built, by Mr. Robert Snyder, with magnetic bearings to reduce friction, had a threshold of 0.09 fps (verified by Hydraulic Laboratory calibration). While this was a considerable improvement over the Ott meter for this specific application, there were still currents in the canals that could not be measured, and in addition the Savonius-rotor has no directional capability.

After experiences with inadequate instrumentation, a Cushing dual-axis electromagnetic current meter, Model 82-CP velmeter sensor with a Model 632-P portable converter, was purchased for the project and provided the necessary low range and directional capability. All velocity measurements by the Hydraulic Laboratory after April 1975 were made with this type of current meter. The measurements provided, for the first time, a detailed look at a current structure which was far more complex than first expected. It was observed that flow reversals are a common part of the hydrodynamic structure of tidal canals, and that even three-layer flows exist. This meter consists of a probe, 3/4 inch in diameter and 11 inches long overall, connected by cable to a signal converter mounted in a hermetically sealed, internal-battery-powered meter box. The probe measures the instantaneous velocity components in mutually perpendicular directions in a plane normal to the

axis of the probe 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 reading 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 in discrete steps 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 provides 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 of full scale.

The probe is extremely sensitive to any movement. A portable tower (Figure 6.2) was therefore designed and built for the Hydraulic Laboratory by Snyder Oceanography Services, Inc., for use in canals. The tower, which can be adjusted in length from 11 to 15 ft, 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 shoreline (Figure 6.3). A carriage holding the velocity probe (Figure 6.4) is free to slide up and down the tower, enabling the probe (Figure 6.5) to be set at any desired depth. With the probe oriented vertically both longitudinal and transverse current components can be continually monitored. The probe must also be oriented 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. Monitoring of the meters is performed on the bank of the canal or on the work boat. Output signals of ± 5 VDC full scale are provided for connection to a strip chart recorder or a data acquisition system.

In practice, a vertical velocity profile is obtained by taking readings at the surface and at 1 ft intervals to the bottom. The carriage is lowered by means of a graduated pole, marked in tenths of feet, which can be secured at any desired depth by means of a threaded wheel. After a longitudinal profile has been recorded, the probe could be turned upward ninety degrees so that the vertical, instead of the transverse, flow component could be measured along with the longitudinal component. This was done at stations near the dead-ends of canals in order to observe vertical currents. Then the probe could be repositioned at the depth of the average current and the output recorded continuously on a dual-channel strip chart recorder.

After some experience had been obtained with a single probe, it was decided that it would be desirable to have a total of five of these systems. This enabled the field crew to take simultaneous readings at up to five locations, or multiple readings at up to five depths. The capability of measuring the velocity distribution across a canal as well as in depth is essential for investigating the nature of secondary flows. In the Loxahatchee River canal study of June, 1977, three towers were placed along the centerline and two to each side at the mouth (Figure 6.6), which permitted an extremely useful set of comparative measurements to be made.

All five probes were calibrated in June, 1977, using the towing facility in the 80 ft flume in the Hydraulic Laboratory. The measurements were recorded on a dual-channel strip chart recorder for later analysis. Runs were made in both directions, over a range of about ± 0.05 to ±1 fps, for each output time constant setting and ranges of 1 and 10 fps. While there were fluctuations of various magnitudes in all the recordings, depending on the settings for time constant and range, there was a combination of settings which would limit the fluctuations to a range on the order of ± 5 percent or less. From each run an average value was then selected by eye. These values, which appear to be accurate to at least ± 1 percent, were plotted against the speed of the towing carriage. At least-squares linear regression and a power curve were separately fitted to each set of data for each direction of flow, the power curve in general accounting somewhat better than the least-squares fit for velocities below 0.1 fps. All data subsequently collected using the Cushing meters, including the Loxahatchee River canal data collected during June, 1977, have been corrected by means of these calibration curves.

6.2.5 Wind Recording

A wind velocity recording set was, unfortunately, not obtained until the fall of 1977, when it was confirmed that wind has a substantial effect on the circulation in canals. Previously, a Davis hand-held low speed anemometer, Model A/2-4", had been used to obtain samples of wind speed. This is a useful meter, but it had the disadvantage that it could be held only about 10 ft off the surface of the water. A measurement taken this close to the water surface is a measurement of a local wind component that 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, was used for long-term wind recordings. Mounted on a ten-meter-high telescoping antenna mast, the sensor was 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 is 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 plexiglass 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 Co. to be better than 5 percent (2.5 mph). The meter was calibrated in the factory before delivery and used immediately upon receipt for the October, 1977, field trip to the 57 Acres site. Thus no additional calibration has been performed by the Hydraulic Laboratory.

6.2.6 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 was found in the results indicated by the two methods, even though the conductivity meter was returned to the manufacturer several times for calibration. After the 9-VDC internal battery had been depleted several times during field operations it was disconnected and supplied from the boat battery through a 12-VDC/9-VDC converter. This still did not result in repeatable readings from the conductivity meter, but it was possible to obtain a calibration curve from sixteen titrations which were performed at the same time as the conductivity measurements on the Loxahatchee north canal. Measurements by titration are accurate to about ±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 (Figure 6.7). 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.

The conduction of an electric current in an electrolyte solution is due to the motion of the dissolved ions rather than the motion of electrons, as in the case of metallic conductors. Thus, this kind of measurement is frequently called "ionic conductivity" or "electrolytic conductivity," measured in micromohos/cm. The electrical relationship,

theoretically expressed by Ohm's Law, is complicated by the fact that the passage of current through the electrodes is accompanied by electrolysis, which may cause both the formation of an insulating film on the electrodes and an increased electrolytic resistance due to removal of ions from the solution (polarization). These effects are large for direct-current measurements [Higgins, 1962, p. 2.4]. Furthermore, air bubbles in the vicinity of the electrodes can affect the conductivity reading, hence the manufacturer's precautions that the probe must be "jiggled" to remove all air bubbles when the probe is immersed.

The titration method is based on a measurement of the concentration of the halide ions [Cl^-, Br^-, and I^-] in the sample. The mass ratio of halide ions to the total volume of water is called the chlorinity. Chlorinity, in ppt, is converted to salinity, also in ppt, by means of Knudsen's formula [1931],

$$\text{Salinity} = 0.03 + 1.805\ (\text{chlorinity}) \tag{6.1}$$

which is incorporated in a correction factor (due to Harvey [1963]) provided with the text kit. Salinity and temperature may be converted to density (seawater only) by means of the *Handbook of Oceanographic Tables* [Naval Oceanographic Office, 1966] or *Tables for Sea Water Density* [Naval Hydrographic Office, 1952].

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 faster 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. Better

conductivity instruments are available, some with automatic temperature correction, but these should always be supplemented by as many titrations as possible to ensure the availability of backup readings in case the conductivity meter does not operate properly or the water is not pure seawater, and for calibration purposes. In addition, a data acquisition system could be used to automatically convert and correct conductivity measurements for greater efficiency of data collection.

6.2.7 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 by the Hydraulic Laboratory to be greater than 2.4°C (corresponding to a density difference of about 7×10^{-4} ppt at 10 ppt salinity and 30°C) in Floridian canals, and since each measurement can be completed quite quickly, both are considered suitable for measurements in canals as long as the readings are taken with the same device at any one station.

6.2.8 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 is, therefore, generally preferred by field crews. The electonic instrument has certain advantages over titration and, once mastered, provides more measurements per unit time. Since only limited numbers of dissolved oxygen measurements have been conducted by Hydraulic Laboratory field crews, almost all by titration, no comparison of results would be meaningful.

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. Since oxygen is likely to be entrained in the sample as it is poured into the empty sample bottle, the field crew should determine a "procedure correction factor" by comparing titrated samples from shallow water taken both directly into a small sample bottle, and by means of the Van Dorn bottle method.

A Yellow Springs Instrument Co. Model 57 dissolved oxygen meter has been acquired for obtaining vertical dissolved oxygen profiles. Calibration of the meter and the probe are straightforward, but depend upon instrument position, atmospheric pressure 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 *insitu* 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.

6.2.9 Dye Concentration Measurements

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 measured, 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 and setting, gives the numerical value of the concentration (except for the location of the decimal point).

The largest concentration the instrument can measure 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 ± 1 percent and its resolution is to ± 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 the concentration of a sample pumped continuously through the machine. The fluorometer has been used successfully in both modes in the field. Figure 6.8 shows a typical set-up of the fluorometer on continuous sampling, connected to a strip-chart recorder, on the work boat.

It has been shown (Section 2.4.6) that the dispersion of a substance introduced as a point source in an unidirectional flow may be

divided into a convective period, during which the movement of the tracer particles is primarily dependent upon their initial convective velocity, and a Taylor or diffusive period in which the movement of the tracer cloud as a whole may be described by the one-dimensional diffusion equation. In an attempt to shorten the convective period as much as possible so that longitudinal dispersion coefficients could be measured over relatively short reaches, two dye injection methods were evolved with the objective of obtaining rapid cross-sectional mixing of an essentially instantaneous source.

The first, and simplest, method employed a pump which forced a mixture of canal water and a predetermined quantity of 20 percent Rhodamine WT dye solution through a hose which was attached to a graduated rod. The rod was positioned at the bow of the work boat so that the end of the hose was oriented downstream at the desired depth, and the entire mixture was dispensed during one traverse of the boat across the canal at the injection station. Typically, at the pumping rate available (about 4 gpm) one traverse could be completed in approximately 40 seconds. The resulting dye source may be considered to be an instantaneous, horizontal line source, and the first part of the mixture typically only traveled about 10 feet by the time the last part had been injected. The dye cloud would be sampled at the same depth as the injection, under the assumption that most of the dye would remain concentrated near that depth.

The second method used a two-stage air pressure system devised by Snyder Oceanography Services. The system provides air pressure from a scuba tank through a check valve to a dye bottle, forcing the dye solution through a hose (Figure 6.9). The pressure in the scuba tank is

reduced through a scuba regulator first stage and a separate, variable 0-125 psi second-stage regulator. The end of the hose may either be positioned, by using a hollow, graduated tube, at a particular depth below the surface for injection from the work boat, or may be connected to a diffuser tube. The diffuser consists of several lengths of 1/2 inch PVC tubing fitted with nozzles spaced at about 10 inches, each having a flow rate of about 5.4 gph at 30 psi. The diffuser tube may be placed transversely on the bed, at the center of the cross-section with the nozzles oriented upward to permit rapid vertical mixing of the dye solution as a plane source. Assuming complete mixing of the dye source in the cross-section, the second method would assure that the tracer would, at least in theory, qualify for analysis by a method based on the analytic solution of the one-dimensional convective-dispersion equation for an instantaneous plane source (Equation 2.6). These injection methods do not necessarily guarantee, however, that the convective period will be shortened. This conclusion will be considered further in Section 6.4.3.

When used at a station to measure the concentration of dye at various depths, it is sometimes convenient to set the fluorometer 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 through a hose and the fluorometer with the continuous sampling cuvette installed. Several electronic instruments for other measurements could be installed in series in such a sampling system.

A sampling system in its simplest form consists of a set of hoses and a pump, as shown in Figure 6.10. It is recommended by the manufacturer that the intake hose be connected directly to the fluorometer, 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. The manufacturer additionally cautions against the use of rubber or vinyl tubing, which tend to absorb minute quantites 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 garden hose is adequate for this application, although there can be some difficulty in maintaining airtight connections. Higher quality commercial hose and quick-connect couplings are planned for future evaluation at the Hydraulic Laboratory.

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. It has been found that it is most convenient to mount the pump on a stand with the fluorometer, so that hoses can be kept out of the way and will not become twisted and kinked. Also, a plug-in arrangement and on-off switch provide the convenience necessary for efficient operation in the boat.

Sampling at a particular depth can be accomplished, if the boat is moored during a test, by simpling placing the necessary length of hose overboard. However, since hose tends to bend and rise upwards, it is

better to have a rigid pipe, such as a length of PVC, which can be placed more precisely at the desired depth. It should also be remembered that the end of the hose takes water in from an elongated volume in front of the intake, which could perhaps extend six inches or more ahead of the pipe. If it is important to take a sample from one particular depth, an elbow should be used at the intake so that water is entrained from the desired depth in a horizontal plane. If any debris is suspected to be suspended at depth in the water, a coarse filter should be installed at the free end of the elbow.

Sampling from a moving boat is somewhat more difficult. PVC tends to bend easily when the boat is under way, and it is difficult to determine the exact depth of intake without measurement. A satisfactory solution for sampling at a depth of 3 ft, using PVC pipe, is shown in Figure 6.11. Later, it was desired to sample simultaneously at two depths, 3 ft and 6 ft. A support on the gunwale of the boat extending down to the waterline, consisting of two thick-walled PVC pipes, was used to hold the inner sampling pipes (Figure 6.12). Both pipes were flexible enough to bend if they hit an obstacle, which was unlikely as the centerline of the canal to be sampled had been thoroughly contoured. The pipes could be withdrawn quickly if necessary up to the water surface, as was necessary a few times when the longest pipe began to bring sand and sediment up from the bottom. A separate pump and simple three-way valve system were used to reverse-flush the long pipe in such instances.

One difficulty with such intake systems may be pump priming. This is usually not a problem with good air-tight fittings and short hoses, but must be taken into account in the overall design of the sampling system.

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 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. This is also an excellent test for proper operation of the fluorometer.

It is possible to set the sensitivity of the instrument in the laboratory on a known concentration of Rhodamine WT dye (10^{-9}, the mid-scale value, is recommended for this instrument). In the field, however, it is still necessary to "blank" the instrument with background canal water and to set the span adjustment each time the instrument is turned on, since background concentrations vary with both location and time throughout a complex canal system (as illustrated in the plots of measured dye concentration in October, 1977, at the 57 Acres system, Appendix B). This can be accomplished by having two large containers available in the boat that are opaque to sunlight (to minimize photodecomposition), one containing a test concentration of dye such as 10^{-9}, and one with initially uncontaminated canal water. The intake and exhaust hoses may then be placed in the containers and the contents of the containers alternately circulated through the instrument during standardization. It is necessary when performing these operations that the hoses be pumped dry before being switched from the canal to the container, and vice versa. Alternately, the fluorometer can be set up for single cuvette samples, in which case the standardization process is considerably simpler.

6.3 Reduction and Analysis of Field Observations

The data collected during preliminary site investigations and field

surveys require extensive reduction before they are in a form that is convenient for the canal designer. Since there is not any particular form that is best for presenting results, data reduction is usually an iterative process involving the canal designer, the field survey team leader and the numerical modeler. The objectives of the field survey will not have been completed until the data have been reduced, analyzed and included in a report which presents the survey techniques, the data and the information gained from the analysis.

Most data have to be converted from values obtained by direct measurement, and combined together in various ways to obtain a useful picture of the variations of some important physical variable in space and time. For example, the salinity distribution in a canal may be derived from measurements of conductivity and temperature, and the designer may decide that he would rather work with density instead of salinity. Furthermore, it is often necessary to group the data together in time and space in a way which is conducive to analysis for cause and effect relationships. For example, grouping of salinity, water velocity and dye concentration for each ebb and flood tidal period aids the analyst in relating the salinity gradients and dye contours to the prevailing and preceeding velocity profiles.

6.3.1 Reduction and Presentation of Tidal Data and Bathymetry

If the water surface elevation has been surveyed and the time of the survey marked on the tide gauge, the tide chart can be easily referenced to MLW. If these tides are then tabulated against the times and heights of high and low tide from the National Ocean Survey (NOS) Tide Tables, one can relate the tides at the site to a reference that will be useful for future predictions. It will be found, provided there have

been no storms or unusual conditions at the time of the measurements, that tides can be predicted at the site to about ± 15 minute accuracy by this procedure.

Depth recordings should always be referenced to time and to location in the waterbody. A measured depth should also always be marked on the depth recorder so that the readings may be calibrated, and all surveys should be run at a constant speed between specific locations which can later be identified on an accurate map or diagram of the plan view of the canal. Then the depth recording is reduced by dividing the reach into equal distance steps and correcting the recorded depths to MLW according to the depth calibration and the tidal elevation.

6.3.2 Reduction and Presentation of Current Readings

Readings of longitudinal and transverse velocity components may either be recorded on a dual-channel strip-chart recorder or logged on a prepared form. Readings are corrected in accordance with previously determined calibration curves.

The results of vertical velocity profile measurements are conveniently presented in the form shown in Figure 6.13. By using this form, both the plan and elevation views of the velocity profiles are presented together so that the directional characteristics of the currents can easily be seen. In addition, by plotting the vertical profile in semi-logarithmic form, it can immediately be seen whether the lower part of the profile is logarithmic. If so, by fitting a straight line to those points which lie reasonably close to a straight line, one can then pick two points from which the bed shear stress can be directly calculated, as described in Morris, Walton, and Christensen, [1978, Section 13.4.3].

6.3.3 Reduction and Presentation of Wind Data

Strip-chart recordings of wind data are normally read at half-hour intervals, with some visual averaging. The data are tabulated and may then either be presented on a hand-drawn or computer-drawn graph, as appropriate for the use of the canal designer. For verification of the numerical model on an existing canal system, the half-hourly wind data may be punched directly on cards for input to the numerical model.

6.3.4 Reduction and Presentation of Salinity Data

Salinity found by titrating water samples is ready for plotting directly, while conductivity must be converted to salinity by using the tables of conductivity as a function of temperature (in five-degree-C increments) and salinity (in 1 ppt increments) provided with the LaMotte conductivity meter. However, these tables are limited to a maximum of 30°C. They are also somewhat inconvenient to use in that, to obtain values to the nearest 0.1 ppt, a three-way interpolation within the table must be performed. This data reduction was efficiently accomplished by a computer program based on an extrapolation of the LaMotte salinity tables [Morris, Walton, and Christensen, 1978, Appendix C.]

The average density of seawater varies between 1.0222 and 1.028 g/cm^3. It is a function of temperature, salinity and pressure, (Figure 6.14) which are readily converted to density by means of the tables referenced in Section 6.2.6. It is often convenient to express the density in terms of the oceanographic variable "sigma-t"

$$\sigma_t = 1{,}000\ (\rho - 1) \tag{6.2}$$

where

ρ = density of seawater, (M/L^3)

This conversion increases the numerical value of the density and thereby makes it easier to work with.

Vertical distributions of density (or salinity) should be plotted together with water temperature (if temperature gradients are significant), velocity profiles, and the longitudinal wind component since they are all indicative of the circulation in the canal. If simultaneous dye concentration data are available, a plot of salinity, velocity and dye profiles will be useful in interpreting the movement of the dye. Figure 6.15 is an example of one of these comparative graphs.

6.3.5 Reduction and Presentation of Dye Concentration Data

Interpretation of the results of dye studies is complicated and limited by the characteristics of the tracer itself, its interaction with its environment, the complexity of the water movement in the canal, and by the small amount of sampling that can be accomplished in the flow field in a limited time. Whether the data are in the form of discrete samples or continuous recordings, their reduction basically involves the conversion of raw data into absolute concentration measurements and the correction of the results for the effects of background and decay.

Rhodamine WT water tracing dye has been widely used for a variety of investigations into the movement of water and contaminants for over a decade. It is generally preferred for this application over other dyes, such as fluorescein and Rhodamine B, because of its greater stability (higher percentage of recovery over time) and less tendency to become absorbed on or absorbed by other materials in the water. Rhodamine WT is a solution containing 20 percent (by weight) of Rhodamine B dye mixed with sodium hydroxide and sodium chloride [personal communication with DuPont Laboratories, April 1, 1975]. It is sold only in solution form,

and it was approved in 1966 for use as an alternative to Rhodamine B by the U.S. Public Health Service [Turner, G. K. and Associates, 1971, p. 3] based on the results of toxicity tests. Wilson [1967] provides much practical information about fluorescent dyes and fluorometry.

All flurescent dyes undergo photochemical decay when exposed to light. Prior to 1975, when the results of an experiment by EPA at Atlantic Beach, North Carolina, were published [EPA, 1975b], no specific test on the decay rate of Rhodamine WT appear to be available in the literature. The data given for Rhodamine B by Feuerstein and Selleck [1963] were therefore used by many investigators for Rhodamine WT. Feuerstein and Selleck state that the concentration of Rhodamine B is a function of: temperature (substantial effect); salinity (slight effect); PH (nearly independent in range 5-10, significant outside this range); background level (naturally occurring substances, possibly significant); and turbidity or suspended solids (significant).

The rate of photochemical decay of Rhodamine WT is a function of the incident solar radiation over the range of photochemical response of the dye during the experiment. The total amount of light reaching the dye is, in turn, a function of location, season, cloud cover, atmospheric particulate concentrations, and turbidity of the water above the dye. EPA [1975b, p. 210] attempted to measure the decay coefficient of Rhodamine WT during one of their canal dye studies in September, 1974. For this experiment clear glass bottles containing known concentrations of the dye were suspended at mid-depth, and measurements of sunlight intensity were recorded at the surface and at mid-depth by means of a marine photometer. Simultaneous recordings of total solar radiation were obtained with a pyrheliometer, which was calibrated in microein-

steins per square meter. The decay rate was found to be 1 percent for "exposure to 308,000 microeinsteins per m^2 of light energy with a wavelength of 590 nm (the excitation length of the tracer)" [EPA, 1975b, p. 210]. Figure 6.15 shows the measured photochemical decay of Rhodamine WT at Atlantic Beach, North Carolina, during EPA's September 1974 field survey. For the EPA field survey the measured decay rate at mid-depth was found to be -0.1278 per day, which was determined to be equivalent to a bright-sunlight (water surface) decay coefficient of -0.534 per day (half life 130 hrs). This agrees well, as observed by the EPA authors, with the decay rate of -0.0224 per hour for Rhodamine B reported by Feuerstein and Selleck [1963, p. 16] when the latter figure is used for a 24-hour period.

An "einstein" is a unit of light energy used in photochemistry, equivalent to Avogadro's member, N_A (6.02252×10^{23} mol^{-1}) multiplied by the energy of one photon of light at the frequency of interest, E_γ. Since the sensitivity of Rhodamine WT peaks at a wave length of 590 nanometers, the peak frequency of interest is

$$F = \frac{\text{velocity of light}}{\text{wavelength}} \tag{6.3}$$

$$= 3 \times 10^8 / 5.9 \times 10^{-7}$$

$$= 5.085 \times 10^{14} \text{ Hz.}$$

The energy of a photon is given by Planck's constant, h (6.6256×10^{-34} Joule sec) multiplied by the frequency, or in this case

$$E_\gamma = hf \tag{6.4}$$

$$= 6.6256 \times 10^{-34} \text{ Js} \times 5.08 \times 10^{14} \text{ sec}^{-1}$$

$$= 3.369 \times 10^{-19} \text{ J/photon @ 590 nm}$$

where h = Planck's constant, [Js]

Thus, one einstein is equivalent to

$$\begin{aligned} Ei &= N_A E_\gamma \\ &= 6.02252 \times 10^{23} \times 3.369 \times 10^{-19} \text{ J} \\ &= 2.029 \times 10^{5} \text{ J} \end{aligned} \tag{6.5}$$

Converting Joules to calories, expressing the energy as a flux (per m^2) and converting einsteins to microeinsteins results in the expression

$$1 \frac{\mu Ei}{m^2} = 2.029 \times 10^5 \text{ J} \times 0.2389 \frac{\text{cal}}{\text{J-m}^2} \times 10^{-6}$$

$$= 0.04847 \frac{\text{cal}}{m^2}$$

Finally, converting calories per meter squared to calories per centimeter squared (langleys)

$$1 \frac{\mu Ei}{m^2} = 0.04847 \frac{\text{cal}}{m^2} \times \frac{1}{10^4} \frac{m^2}{cm^2}$$

$$= 4.8472 \times 10^{-6} \frac{\text{cal}}{cm^2}$$

$$= 4.8472 \times 10^{-6} \text{ ly at 590 nm} \tag{6.6}$$

This conversion factor was confirmed by EPA in a personal communication with the writer during March 1978. However, it is recognized that this conversion factor has been calculated for only one wavelength component of the solar radiation, and therefore cannot be used directly to obtain the photochemcial decay rate for Rhodamine WT. Knowing that the visible light spectrum peaks approximately at 590 nm, the measured solar radiation in the visible light spectrum (400-700 nm) could be converted back to total solar radiation (which was recorded by the pyrheliometer but not reported by EPA) to obtain the rate of decay of Rhodamine WT in terms of total incident solar radiation. The calibration curves

(similar to those given by EPA [1973, p. XII-5]) for the photometer and the pyrheliometer would have to be used to obtain this conversion.

The order of magnitude of measured photochemical decay at mid-depth in the North Carolina canals has been found to be significant. It is therefore recommended that during any dye dispersion experiment which is to be conducted over a period in excess of one day, a sample of the dye be moored at the sampling depth and its concentration measured daily during the field survey. G. K. Turner has pointed out that soft glass shifts the spectral peak of Rhodamine WT, whereas pyrex transmits most of the incident light without distortion [personal communication with G. K. Turner, Feb 1978]. Until measurements are available, it is therefore recommended, that a pyrex container be used for the test dye sample. If this procedure is used, the effects of atmospheric attenuation, water temperature, and turbidity on the incident solar radiation will be automatically taken into account when periodic measurements of dye concentration are taken from the submerged container.

Feuerstein and Selleck [1963, p. 5] found that the temperature of a sample significantly affects the Rhodamine B analyses. They recommend that all readings be corrected to a standard temperature, assuming that the concentration is a linear function of the fluorescence, according to the expression

$$\frac{c}{c_o} = \exp\,(-0.027t) \tag{6.7}$$

where

c = concentration to be found, (dimensionless)

c_o = concentration at reference temperature, (dimensionless)

t = temperature above reference temperature, (°C)

Since the water temperature in the canals studied by the Hydraulic Laboratory was typically constant at the sampling depth, within one or two degrees Celcius, it has not been considered necessary to apply a temperature correction. Likewise, no correction has been incorporated into the dye concentration data for photochemical decay, since this decay can be incorporated in the model (CANNET3D) in the form of a rate constant.

To assist in analysis, the reduced dye concentration data may be plotted either as vertical concentration values (for example, see dye concentration data in Figure (6.15)), as vertically averaged profiles (Figure 6.17), or as continuous profiles for each reach as in Figure (6.18), depending on whether they are point measurements or continuous measurements. The latter plots may be obtained by using the computer program documented in Morris, Walton, and Christensen [1978, Appendix C].

A longitudinal dispersion coefficient may be calculated from measurements of concentration versus time at a particular location near the end of a canal reach. The modified semi-log plot method [Holley and Harleman, 1965, p.110] was used by the Hydraulic Laboratory for the 1975 longitudinal dispersion studies. This method is based on the analytical solution to the one-dimensional convective dispersion equation for a constant mean velocity and an instantaneous plane source, Equation (2.6). Use of the modified semi-log plot method is based on the following assumptions:

a. The flow is steady and uniform, i.e. that the time-mean, cross-sectionally and longitudinally averaged current $\bar{U}$ is representative of flow during the entire measurement period.

b. The source is an instantaneous, uniform plane source. From a plot of the concentration versus time, at a point at or near the peak concentration, c_*, is selected for the modified semi-log method. For the corresponding time t_* equation (2.9) may be written with c_* substituted for c and t_* substituted for t, and after dividing, rearranging, and taking the natural log, the following relationship is obtained:

$$\ln\left[\frac{c}{c_*}\sqrt{\frac{t}{t_*}}\right] = -\frac{1}{E_\ell}\left[\frac{(L - \bar{U}\ t_i)^2}{4t} - \frac{(L - \bar{U}\ t_*)^2}{4t_*}\right] \tag{6.8}$$

where

c_* = concentration at or near peak, (dimensionless)

t_* = time corresponding to c_*, (T)

$\bar{U}$ = L/t_p, (L/T)

L = distance from location of dye injection to location of concentration measurement, (L)

t_p = elapsed time to measurement of peak concentration, (T)

When the two bracketed quantities are plotted versus each other for the concentration distribution at distance L from the dye release point the data will be on two relatively straight lines, one for the reading on the rising limb of the concentration curve and one for the falling limb. The slope of the bisector of these two lines, which intersect at the peak value, is $-1/E_\ell$ and the dispersion coefficient E_ℓ may be readily determined.

A further simplification may be achieved by substituting tp for t_* where t_p is the elapsed time to peak ($L/\bar{U}$). If c_* is selected as exactly the

peak concentration, equation 6.8 becomes

$$\ln \left[\frac{c_i}{c_*}\sqrt{\frac{t_i}{t_*}}\right] = -\frac{1}{E_\ell}\frac{(L_d - Ut_i)^2}{4t} \tag{6.9}$$

where

c_i = selected concentration values on the concentration curve, (dimensionless)

t_i = time from injection to each selected concentration value, (T)

There are two convenient features of the modified semi-log plot method. First, the determination of E_ℓ is independent of the value of the injected mass M, as can be seen in equation 6.8, provided that the mass M is large enough to produce a sufficiently large change in concentration at the sampling point, and not so large as to exceed the range of the fluorometer or to saturate the banks of the waterbody and cause significant secondary peaks. Secondly, the analysis is completely objective once points have been chosen on the concentration curve.

6.4 Field Observations in Floridian Canals

The task of analyzing the data which have been collected during the canal design research project has been spread out over the entire three years that the study has been in progress. At the beginning of the project, the work concentrated on measurements of the variability of key parameters in typical canals around the state of Florida, on some preliminary design analyses, and on finding the ranges of the longitudinal dispersion coefficient in Floridian canals. After these parameters had been reasonably well quantified, more experience with numerical modeling and a better understanding of the circulation of

water in more complex canal networks were undertaken as intermediate objectives. Finally, the details of wind-induced and density-induced flow and dispersion were investigated in support of the final development of the three-dimensional model. This section summarizes the results of these field surveys and shows how the conclusions developed from them influenced the design of the numerical model and the canal design techniques.

The preliminary site investigations and field surveys conducted by the Hydraulic Laboratory from 1974 through 1977 are summarized in Table 6.2. The author, acting as survey team leader on all of the field surveys undertaken (except Venus Waterway), provided continuity for the research. The locations of all survey sites are shown in Figure 2.13.

Variability studies at Port Charlotte and Punta Gorda, on Florida's west coast and at Loxahatchee River and Pompano Beach on Florida's east coast during 1975 were planned primarily for obtaining the measurements of tidal ranges, canal geometry, salinity, water temperature, and dissolved oxygen which were used by Bailey [1977] in his statistical analysis of Floridian canals.

Design modification and improvement surveys were undertaken at Cudjoe Gardens in the Florida Keys in 1974, at Venus Waterway, Charlotte County, in 1975, and at the 57 Acres canal site in April 1976. The purposes of these surveys were to obtain experience in analyzing the flow and dispersion characteristics in such canal systems, and in evaluating the possible design alternatives available for improving these systems. The location of Cudjoe Gardens is shown in more detail in Figure 6.19, while Figure 6.20 is a sketch of the location of Venus Waterway. The location of the 57 Acres site is included in Figure 2.15. These studies are summarized in Section 6.4.2.

Field studies to determine the magnitude of the longitudinal dispersion coefficient were performed in 1975 at the Frenchman's canal site and at the 57 Acres site (Figure 2.15), both connecting to the Intracoastal Waterway (ICW) south of Jupiter, Florida. The former is a simple, Z-shaped canal with straight reaches, while the latter is a complex network with two tidal entrances and numerous curved canals. In addition to the specific measurements of longitudinal dispersion, preliminary site investigations were conducted in preparation for the fluviohydrographic survey, and the comprehensive field surveys were conducted at these sites. This field work is described in Section 6.4.3.

The *comprehensive field surveys* were undertaken in 1977 at the Loxahatchee River north canal (Figure 2.14) and the 57 Acres Canal System. The objectives of these surveys were to obtain a better understanding of the effects of winds, salinity gradients, and bends on the flow in canals and to provide detailed measurements of these effects for use in verifying the numerical model. These studies are summarized in Sections 6.4.4 and 7.5.2.

6.4.1 Variability Studies

The variability studes were conducted in support of a project conducted by the Department of Environmental Sciences and Engineering at the University of Florida. The results are summarized in Table 2.2 and in Bailey [1977]. In addition to collecting data for variability analyses, it was suspected that some canals would have significant density gradients, while others would not. Some degree of stratification is usually found in a canal if the receiving water consists of an estuary which is not well-mixed, such as one which has a significant flow of river water superimposed on tidal fluctuations. Therefore,

salinity and temperature profiles were measured in September 1975 in one of each of the four pairs of canals at Port Charlotte, Punta Gorda, Pompano Beach, and Loxahatchee River [Hydraulic Laboratory, 1976].

At the time of the measurements all of the canals exhibited some degree of salinity stratification. The data for the Loxahatchee and Punta Gorda canal showed that a salinity wedge occupied the entire length of each canal on a flood tide. The Port Charlotte canal had a relatively steep but uniform salinity gradient at all locations in the canal on an ebb tide, and on the flood a wedge could also be detected. The Pompano Beach canal showed the presence of a remnant saline dome inside the canal on a flood tide, and relatively uniform salinity gradients at high tide [Hydraulic Laboratory, 1976]. It was concluded from this limited number of samples that stratification does occur in canals on both coasts of Florida. Whether this is a common feature or not could not be concluded from the limited number of measurements.

6.4.2 Design Modification and Improvement Surveys

In October, 1974, a fluvio-hydrographic survey was conducted by the Hydraulic Laboratory at Cudjoe Gardens, Cudjoe Key. Its objectives were to predict the overall effect of completion of canal construction in a portion of the site which had been previously dredged, but was still plugged, and to predict the hydrographic effects of constructing a connecting link between the two main sections of the canal network. Accordingly, measurements of bottom profiles, tides, currents, and water quality were made and an analysis of these measurements and the general operation of the system, both before and after interconnection, was completed. The results of this work are reported in Morris and Christensen [1975].

The Cudjoe Gardens canal network is shown in Figure 6.21. Its arrangement at the time of the survey comprised two isolated sections, one consisting of canals A through J and the second consisting of canals K through N, with some additional isolated canals connected directly to Bow Channel. Canals K through N were located in the plugged section, the plug being located on Bow Channel near the intersection of canals K and M. The proposed interconnection between the two isolated portions was planned for the west end of reach I.

At the time of the analysis a realistic numerical model for canal networks was not available. Calculations based on the data from the field survey indicated that the tides at the two entrances would have enough phase difference to cause a significant increase in the tidal circulation upon the removal of the plug and interconnection of the two systems. It was suggested that an oscillating null point would be established near reach H, and that flushing would be improved, not affected adversely.

This survey and analysis provided a useful insight into field measurement problems and canal network operation. It showed that questions on hydrodynamics can be addressed rationally and analyzed logically by basic hydraulic principles.

Venus Waterway and its tributaries, which are connected to the Myakka River near Port Charlotte, were the subject of two fluvio-hydrographic surveys by the Hydraulic Laboratory in March and May, 1975. The objective of the study was to determine the effect of dredging an interconnection between the principal part of the canal network and a small section called the Terminus at a distance of about 900 ft from the end of the main canal (Figure 6.22). Measurements of bathymetry, tidal

elevations, current velocities, DO, BOD, other water quality components, and dispersion coefficents were made. It was concluded that the increase in tidal prism would result, temporarily, in increased scour as the system adjusted itself over a period of time to a new cross-section. The removal of the sill at the outlet of the terminus, compled with the increased tidal prism, would improve vertical mixing in the Terminus, and flushing at least in the main channels.

This survey and the subsequent analysis provided additional experience in field measurements. The dispersion coefficients obtained from the field studies confirmed values obtained in other parts of the State. Some general guidelines relating to the role of the tidal prism in flushing canal networks were advanced, but a numerical model was not available at that time to quantify the effects of changes in the geometry of the canals or the tidal prism.

The 57 Acres site was the subject of a variety of field surveys, including fluvio-hydrographic surveys to provide basic measurements for the analysis of circulation, bed and bank stability, and flushing in the network. During 1975 and 1976, measurements of vertical velocity profiles and longitudinal dispersion coefficients were completed in every major reach in the network. Following the survey conducted during the period of April 1 to 4, 1976, tidal-prism analyses was conducted for the existing network, and for a modified version with additional tidal inlets and interior lakes designed by R. M. Snyder [1976], as reported in Walton et al [1975b].

During this set of surveys the basic instrumentation requirements and measurement techniques for canal research were established and tested. In addition, fundamental objectives for the numerical model

were established and the limits of the prediction of flushing by tidal prism analysis were reached. The hydrographic studies completed during 1975 and 1976 formed a basis for the comprehensive field surveys conducted during 1977 (see Section 6.4.4.2).

6.4.3 Longitudinal Dispersion Studies

One of the first problem areas defined in the canal design research program was the lack of data on diffusion and dispersion in tidal canals. A review of the literature on diffusion and dispersion revealed that much has been accomplished in terms of quantifying dispersion in undirectional flow (e.g. in rivers [Fischer, 1967b]) and that some work had been done to measure diffusion in estuaries [Hetling and O'Connell, 1965], but it was suspected that the geometry of the waterbody would have a substantial influence on the rate of spread of a pollutant and therefore plans were made to undertake some dispersion studies in canals in early 1975.

As was shown in Section 6.35, there are a number of different ways of analyzing tracer concentration profiles to obtain a longitudinal dispersion coefficent. Since each requires a particular set of measurements, it is necessary to decide before going to the field exactly what mesurements the canal designer needs. The modified semi-log plot method was used by the Hydraulic Laboratory in 1975. This method, while somewhat oversimplified, provided consistent results and was preferred because it could be used in the future by relatively untrained field crews to obtain order-of-magnitude values for the coefficients.

Longitudinal dispersion experiments were conducted in the Frenchman's Canal and at various locations in the 57-Acres canal network between May 10 and November 3, 1975. The geographical locations of these

canals have been shown in Figure 2.15. The reach connecting the intracoastal waterway with the North Bend of Frenchman's canal, in which the dye dispersion measurements for that canal were conducted, is approximately 2000 ft long and is shown in Figure 6.23. The layout of the 57-Acres canal network, and the locations associated with the 1975 dispersion studies, are given in Figure 6.24.

It was known that the entrance reach in the Frenchman's canal was too short for a point source of tracer to become well-mixed, with enough distance remaining for a reasonable experimental ditance. Likewise, it was realized that the straight reaches in the 57 Acres canal network were similarly limited. Consequently, the pressurized tank method described in section 6.2.9 was developed and for all dispersion experiments the dye was injected into the surface layer of the canal and mixed across the cross-section.

The intake end of the sampling hose was positioned at the designated depth with an anchor line and float, and the fluorometer was calibrated, before the experiment was started. The boat was then driven to the desired injection location, the dye was injected, the boat was driven slowly back to the sampling point, and the pumping system was connected to the intake hose. The fluorometer was set to the desired range and used either in manual mode (if the peak concentration could be predetermined) or in automatic-ranging mode.

A two-pen recorder indicated both the range of the fluorometer and the value of concentration. A record of the following was kept for each experiment:

Distance from injection to sampling point

Total depth and injection depth

Average current velocity

Average wind speed and direction

Mass of dye injected

In the auto-ranging mode, the operator must usually exercise some manual control over range switching to be certain that excessive switching does not occur and to decide when to terminate the experiment. A typical dye study in a canal, performed over a distance of 800 feet with a current of 0.5 fps, will usually take about 45 minutes to an hour depending on the rate at which the concentration decreases after the peak.

The recorded concentration curve was not analyzed until after the crew returned to the Laboratory. Then, the record was reduced to a single curve by applying the instrument range ractors to discrete values at two to four second intervals from the recording and processed by a computer program which finds the value of E_ℓ by the modified semi-log method.

The results of the longitudinal dispersion studies are summarized in Table 6.3. In general, it was found that the dimensionless dispersion coefficients K were very low, in the range of 2 through 20, in comparison to values of from 50 to 700 found by Fischer [1967b, p. 188] in natural rivers.

6.4.4 Comprehensive Field Surveys

Three comprehensive field surveys were conducted in 1977 to study the effects of winds, salinity gradients, and bends on the flow in canals, and to obtain data for calibration of the numerical model. A general discussion of these surveys is contained in this section, while a more detailed discussion of those parts of the results which pertain to verification of the numerical model will be found in section 7.5.

6.4.4.1 Loxahatchee River Field Survey

The first of the comprehensive field surveys was undertaken at the Loxahatchee North canal from June 10 to 16, 1977. It had been observed on previous field surveys that these canals were frequently characterized by two- and three-layer flows, and it was suspected that these circulation patterns might have been caused by winds and/or density gradients. Accordingly, the hydrographic survey plan included the measurement of vertical velocity profiles, the collection of salinity, temperature samples and dissolved oxygen samples, and a dye dispersion study over a three-day period in the North canal.

The specific objectives set forth in the survey plan were to:

1. determine the longtiduinal dispersion coefficient and flushing characteristics of the canal.
2. determine dye interchange at the mouth between flood and ebb tides.
3. Measure vertical salinity, temperature, and velocity profiles simultaneously to obtain data useful in describing the interrelationships among these variables.
4. Measure wind to obtain data on the relationship of wind to vertical velocity.
5. Obtain data for verification of the numerical model.

The field crew had never worked together in the field before, and only the team leader had had experience in hydrographic surveying of canals.

A recording tide gauge was placed on the north side of the canal inside the mouth, and five current meter towers were installed, three along the centerline, and two inside the mouth in line with a tower on the centerline (Figure 6.25). Two work boats and a crew of four were

available for daytime work. Stations at 100 ft intervals, visible from the boats, were established on shore and the bathymetry of the canal was obtained.

Measurement of dye concentration profiles, for determination of the longitudinal dispersion coefficient, was planned for two locations along the length of the canal so that the change-in-moment method [Fischer, 1967b, p. 198] could be used to determine the coefficient. Samples of dye were to be taken at three depths on the centerline and at a 3 ft depth on each side of the centerline, at each station, by means of an assembly of PVC pipe, flexible tubing, and pumps which had been built and tested in the laboratory prior to departure. Unfortunately, this system had never before been tested in the field and its assembly at the site proved to be excessively difficult. In addition, the sampling arrangement for the boats could not be operated efficiently, and it was decided to revert to measuring dye concentrations in water samples taken sequentially at one foot depth intervals at stations along the centerline. Salinity, temperature, and occasionally dissolved oxygen, were measured from the same samples.

After the dye had been injected by means of the pressure tank described in Section 6.2.9, one boat was assigned the task of taking current velocity and wind speed readings while the other proceeded to obtain water samples and measure conductivity, salinity, temperature, dye concentration and DO. These measurements were recorded on log sheets for reduction in the laboratory. The reduced measurements are presented in Appendix A. Section A.1 contains a summary of the dispersion study, a dimensional drawing of the canal showing the locations of the tide gauge and velocity stations, and a summary of the 3-hour

climatologic data from West Palm Beach airport for the month of June. Section A.2 contains a series of 8 sets of graphs of salinity, velocity, wind, and dye concentration versus depth and location in the canal. The measurements were grouped within each ebb and flood tidal half-period and plotted together. In cases where more than one set of measurements of a particular variable were taken during the same half-tidal period, or in cases where the measurements were taken at substantially different times, more than one set of graphs were prepared for the same half-tidal period. For example, there are two sets of plots for the ebb tide beginning at 0800 on June 14, one for the period 1043 to 1259, and another for the period 1424 to 1603.

Winds were measured with a hand-held anemometer along the centerline of the canal. It was concluded, however, that these measurements were of little value since they were taken below the tree canopy and were too infrequent to provide a wind history for modeling pruposes.

The measured longitudinal velocity profiles showed the existence of one-, two-, and three-layer (alternating) flows, many taking the form of the basic wind-induced velocity profile (Appendix A, Section A.2). Several vertical velocity profiles were measured near the dead-end, but the vertical components were quite small (on the order of 0.05 fps) and fluctuating so that it was not possible to discern a sustained flow either upward or downward.

However, when considered in conjunction with the vertical salinity profiles, the presence or absence of a salinity wedge or dome and its approximate location in the canal could be observed. For example, Figure 6.26 shows the presence of a density wedge near the end of an ebb tide, where the interface between the greater-density water mass and

the less-dense water mass is indicated by a significant change in the salinity gradient. As mentioned previously, the measured change in temperature with depth in the Loxahatchee canal at the time of these measurements was insignificant and the salinity gradients are considered to be representative of the density gradients. The elevation of this salinity wedge decreases with distance into the canal, but its presence is indicated even at the station located 150 ft from the dead-end. Near the entrance the denser mass reaches its peak elevation, and at the entrance it has become "rounded off", which is a common feature that occurs on an ebb tide.

In contrast, Figure 6.27 shows a less pronouned set of salinity gradients just after the beginning of a flood tide. These would appear to indicate the beginning of formation of a saltwater wedge, the effect of which can be seen to reach the length of the canal. Figure 6.28, showing measurements taken approximately at mid flood tide, indicates the presence of a remnant salt dome from a previous tide near the dead-end, and no indication of intrusion of a new salt wedge. There is, unfortunately, not enough additional information in this case, or for that matter in any of the measurements obtained during this field survey, to explain the conditions causing the presence or absence of salt wedges or to quantify their rate of formation and movement.

The dispersion study performed in conjunction with this field survey provided data which could be used to verify the numerical model, which is described in some detail in Section 7.5.3. A volume of 75 ml of Rhodamine WT dye, 20 percent solution, was released and mixed across the channel in the surface layer at 200 ft from the dead-end at four hours after high tide. The vertical profiles of dye concentration at

100 ft intervals along the centerline (Appendix A, Section A.2) are quite complex and cannot be reduced to meaningful contour lines. For example, Figure 6.28 shows the concentration values about four hours after dye release, at the beginning of the flood tide. The concentrations are still quite high near the dead-end, since there had not been much flushing during the previous ebb. Concentrations are higher around mid depth, due to the outward wind-induced flow in the surface layer, and they generally increase toward the dead-end. Figure 6.27 shows the concentration values two days later, after which time they have been significantly reduced. Vertical averaging of such profiles (Figure 6.17) permits a simplified analysis of flushing time to be made with a calibrated model.

It will be shown (Section 7.5.3) that the data obtained during this field survey was useful for verifying the model, in spite of their apparent limitations. While most of the remaining specific objectives were only qualitatively completed, much was learned about the limitations of field measurements by the new field survey team. This experience contributed substantially to the success of the field surveys to follow.

6.4.4.2 57 Acres Field Surveys

The second and third comprehensive field surveys were undertaken at the 57 Acres canal site in July and October, 1977. Their specific objectives were to obtain data for the calibration of the numerical model. Since a numer of field surveys had already been conducted in that canal network by the Hydraulic Laboratory, it was possible to select an ideal location for measuring vertical velocity profiles and conducting a long-term dye dispersion study. The field crews consisted

of both new and experienced personnel.

The first field survey at the 57 Acres site was conducted from July 18 through 22 with a team of four. The specific objectives of the survey plan were to:

1. Locate a reach in the canal network which would be suitable for a dye study.
2. Measure vertical temperature and salinity profiles for possible density gradient influence.
3. Conduct a dye study over a several-day period to determine the feasibility of day and night field measurements.

The South Loop of the canal network (Figure 6.24) was chosen for the dispersion study because it had a dead-end, was relatively long, did not appear to be influenced by density currents, was accessible from the road for the transport of heavy equipment, and was isolated from boat traffic. Previous bathymetric surveys indicated that its depth was relatively uniform, and a sharp bend at some distance from the dead-end was available for measurements of bend-induced helical currents.

The dispersion study was initiated just after midnight on July 20. Fifteen hundred ml of Rhodamine WT, 20 percent solution, was released as a point source about 400 ft from the dead-end of the South Loop at the surface from a bottle under no-wind conditions at high tide. Dye concentration profiling throughout the canal network began just after the following high tide, and continued on each successive high and low tide for nine half-tide periods (except one low, which was missed due to equipment malfunction). The dye concentration was sampled continuously beginning at the dead-end of the South Loop, from a depth of about

three ft along the centerline. At all locations in the network that were sampled a value above zero concentration was detected, but when the operators decided that they had been measuring a constant value for some time, such as the length of a reach, sampling was terminated in that direction.

The concentration data were recorded on a strip-chart recorder for reduction later in the laboratory. The reduced data were plotted on a two-dimensional graph for each reach, and on a three-dimensional representation of the entire network. Each reach is designated by its two endpoint stations, as shown in Figure 6.24. Thus, reach AD is the South Loop to its intersection with the South Straight. The two- and three-dimensional plots are included in Appendix B.

The three-dimensional plots show the 57-Acres canal system from the northwest. All plots for both field surveys are given with the same horizontal scale, azimuth, and elevation. The northeast-southwest distance scale is 1,000 ft/in while the northwest-southeast distance scale is 2,000 ft/in. The canal network dimensions are represented in accordance with these scales and are not foreshortened. The vertical scale for the first survey is 75 ppb per inch, while it is only 5 ppb per inch for the second. All measured values are given as reduced from the strip-chart recordings without any adjustment for apparent background concentration. Any reach on the graphs or maps with a zero concentration is a location where no measurements were taken.

The nine three-dimensional plots, two representative examples of which are given in Figures 6.29 and 6.30, show qualitatively the following features:

1. The peak concentration moves out of the canal on an ebb tide and back into the canal on a flood tide.
2. In the south loop the peak gradually attenuates and the dye appears to spread both upstream and downstream. However, there appears to be substantial mixing and storage of dye in the dead-end of the south loop, probably from dye being returned by means of the lower layer.
3. There is an overall loss of mass over the forty-three hour test, but not a linear decrease in the peak nor a linear increase in the standard deviation of the concentration distribution. Also, at the beginning of the study, there appears to be generally a greater mass of dye in the surface layer during the ebb tide.

These trends do not follow what would be expected if the flow was predominantly tidal, with no wind and density effects. In fact, on close inspection there are discrepencies which are not easily explained. It must be remembered, however, that the readings were taken only along the centerline of the canal and at a depth of 3 ft, which is in the top layer of a wind-induced flow regime. Unfortunately, neither wind nor velocity measurements were taken during this feasibility study.

Each dye concentration run was conducted with two crew members on a voluntary basis, and scheduling of sleeping time was informal. This was a satisfactory arrangement under the circumstances, but could not have been carried on indefinitely. It was concluded that, while feasible, sampling on every successive high and low tide would require a much larger field crew under more formal conditions.

The second of the two field surveys at the 57 Acres site was conducted from October 16 through 22, again with a crew of four. Two new personnel, one being the numerical modeler who was developing the canal network model, completed the team.

This study included wind and water velocity, salinity, and water temperature profile measurements. As a result of these measurements, density gradients in the 57 Acres canal network were again determined to be negligable at that time. The wind was recorded from October 18 at 1500 to October 21 at noon (Figure 6.31). Velocity profiles were taken at the five stations shown in Figure 6.32 from October 17 to 21, except on October 20 the velocity meters at stations four and five were moved to station three as shown in Figure 6.33.

After several days of taking velocity profiles at five locations in the South Loop, and after two of the velocity meters were moved, the dispersion study began. Again, 1500 ml of Rhodamine WT, 20 percent solution, was released, but this time as a line source across the channel in the upper 3 ft at high tide. Centerline profiles were recorded for the seven subsequent high and low tides. The two- and three-dimensional plots of these dye concentrations are included in Appendix B.3.

In general, it is difficult to even make any qualitative statements about the results of this second dispersion study. The plots for the high tide at 0420 on October 21 (Figure 6.34) and the low tide at 1045 (Figure 6.35) appear to follow the expected trend of a reduction in the mass of dye throughout the system. Furthermore, portions of some of the other paris of high and low, or low and high, plots *not near the dead-end* demonstrate reasonable behaviour. For example, the reach beginning

from a point about 2,000 ft westward of the dead-end of the South Loop, shown during ebb after the high at 771020/0330 (Figure 6.36) and during the following flood after the low at 771020/0950 (Figure 6.37) demonstrates moderate flushing. However, in the same two dye concentration plots (Figures 6.36 and 6.37) in the first 2,000 ft of the south loop the peak moves *toward* the dead-end and the total mass of dye appears to *increase* during the ebb. There are several possible explanations for this observation, which illustrate some of the limitations in this experiment:

1. Inspection of the wind record (Figure 6.31) shows that the surface layer was being blown out of the canal during the entire period of the ebb between 0330 and 0950. Thus, it may be surmised that some dye could have been entrained into the returning (upstream) lower layer.
2. Some of the dye injected at high tide could have been entrained into the residual circulation from the previous flood tide and carried into the dead-end, where it could be stored in lower layers and in eddies which may exist in this extremely unpredictable portion of the canal. Velocity measurements were not taken in this area to assist in resolving this possibility.
3. It is known that the background concentration will change with time and location in the canal system. While a background change is not evident in the two pairs of plots discussed here, its effect might be enough to contribute to the difficulty in obtaining reasonable results.

From the two dye dispersion experiments reviewed in this section, it is evident that when dealing with multi-layered flow and the complexities of the circulation near a dead-end, a sample of dye concentration taken at only one depth along the centerline will not provide an adequate description of the history of dye movement. As a minimum for these conditions, the dye should be sampled at two depths throughout the network, vertical velocity profiles should be measured at the midpoint of each straight reach, and the tracer should be released at a substantial distance from the dead-end to avoid dead-end storage effects. These ideas are discussed more fully in Section 7.5.2.

Table 6.1 - List of Field Equipment and Instrumentation, Which Can Be Used for Each of the Measured Variables in Table 5.2.

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

Table 6.2 - Preliminary Site Investigations and Field Surveys Conducted by the Hydraulic Laboratory (University of Florida) During the Canal Design Research Project.

Location	Dates	Type	Tide	Depth	Current Speed	Current Velocity	Wind Speed	Wind Velocity	Salinity	Temp.	DO	Dispersion	Bed and Bank Material
A. Variability Studies													
Port Charlotte and Punta Gorda	750322-750324	PSI	X	X	X								
	750614-750617-	FS	X	X	X								
	750906-750909	FS	X						X	X			
	751122-751124	FS	X	X									
Loxahatchee River	750325-750326	PSI FS	X	X	X								
	750614-750617	FS	X	X	X								
	750911-750914	FS	X						X	X			
	751116-751118	FS	X										
Pompano Beach	750327	PSI FS	X	X	X								
	750614-750617	FS	X										
	750909-750912	FS	X						X	X			
	751121-751122	FS	X										

Table 6.2 - Continued.

Location	Dates	Type	Tide	Depth	Current Speed	Current Velocity	Wind Speed	Wind Velocity	Salinity	Temp.	DO	Dispersion	Bed and Bank Material
B. Design Modification and Improvement Surveys													
Cudjoe Gardens	741009-741013	PSI											
		FS	X	X	X				X		X		
Venus Waterway	750315-750317	PSI											
		FS	X	X	X				X		X	X	X
	750516-750517	FS	X		X						X		
57 Acres	760401-760404	FS	X			X	X						
C. Longitudinal Dispersion Studies													
Frenchman's Canal	750131-750202	PSI											
		FS	X	X	X				X	X			
	750509-750511	FS	X			X						X	X
	750622	FS	X			X						X	
57 Acres	750622-750623	PSI											
		FS	X	X		X							
	750914-750918	FS	X			X							X
	751002-751005	FS	X			X						X	
	751031-751105	FS	X			X						X	

Table 6.2 - Continued.

Location	Dates	Type	Tide	Depth	Current Speed	Current Velocity	Wind Speed	Wind Velocity	Salinity	Temp.	DO	Dispersion	Bed and Bank Material
D. Cumprehensive Field Surveys													
57 Acres	770719-771923	FS	X			X	X					X	
	771016-771022	FS	X			X		X				X	
Loxahatchee River	770610-770615	FS	X	X		X	X		X	X		X	

Table 6.3 - Summary of Results of Dispersion Measurements by the Hydraulic Laboratory (University of Florida) During 1975.

DATE (1975)	STA.	TIDE	WIND (kn.)	DEPTH (FT) TOTAL	DEPTH (FT) SAMPLE	AVE. VEL. C. of M. (FPS)	DIST. TO SAMPLING STA. (FT.)	LONGITUDINAL DISPERSION COEFF. (FT^2/SEC)	DIMENSIONLESS DISPERSION COEFFICIENT $K = E_L/Ru_*$
I. FIELD TESTS AT FRENCHMAN'S CANAL									
May 10	2	F		6.0	1.0	0.25	335	4.78	41.2
May 10	2	F		6.4	1.0	0.24	335	2.40	18.8
Jun 22	1	E	8(VAR)	7.5	4.0	0.19	290	0.65	2.5
Jun 22	2	F		4.7	0.5	0.28	230	1.80	21.3
Jun 22	2	F		5.0	3.0	0.21	520	0.64	7.6
II. FIELD TESTS AT 57 ACRES									
Oct 3	M	F	10 E	9.1	4.0	0.34	780	3.00	8.0
Oct 4	N	E	10 E	11.0	4.0	0.30	780	1.69	4.5
Oct 4	R	F	10 SE	9.5	4.0	0.55	630	1.99	5.5
OCT 5	Q	E	10 SE	10.3	4.0	0.33	835	2.09	5.8
Oct 31	B	E	10 NE	11.0	3.0	0.17	800	1.01	2.6
Oct 31	C	F	10 NE	10.0	3.0	0.46	800	1.09	2.9
Nov 1	U	E	15 NE	8.9	3.0	0.23	570	1.84	5.9
Nov 2	S	E	10 NE	8.5	3.0	0.41	660	4.74	21.6
Nov 2	T	F	10 NE	8.1	3.0	0.29	660	3.58	16.3
Nov 3	E	F	10 NE	9.4	3.0	0.34	650	0.63	8.1
Nov 3	F	E	10 NE	0.4	3.0	0.20	650	0.63	8.1

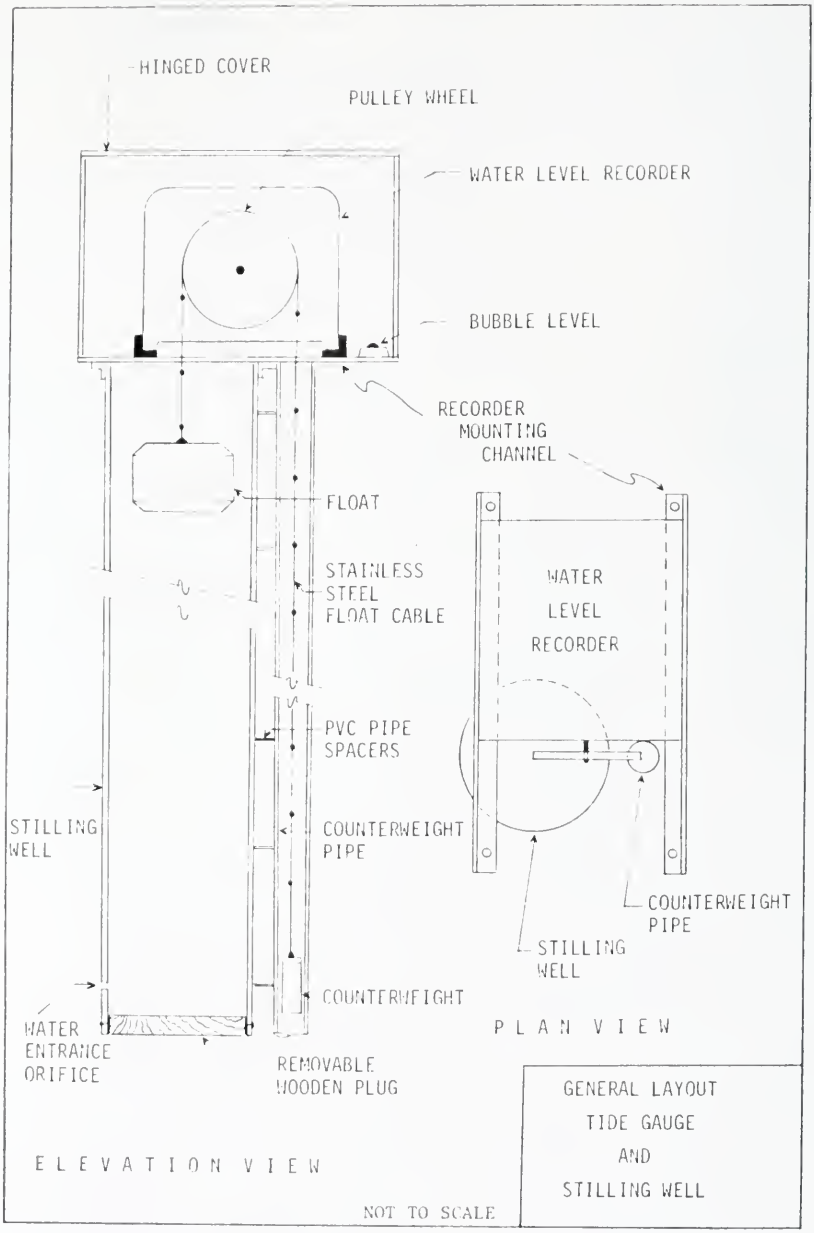


Figure 6.1 - Typical Stilling Well and Water Level Recorder Box for the Tide Measurements.

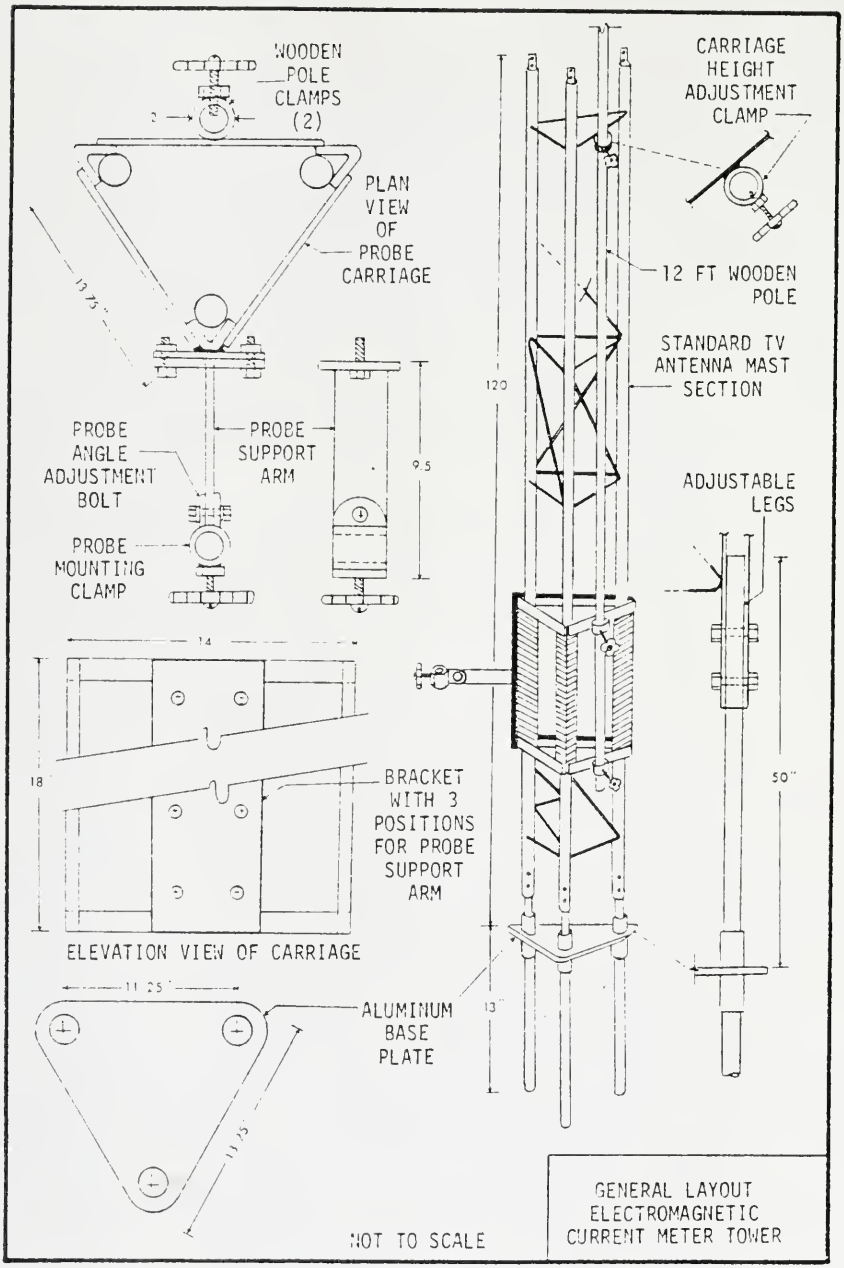


Figure 6.2 - Dimensions of Velocity Meter Tower with Adjustable Carriage, Designed and Built by Snyder Oceanography Services.

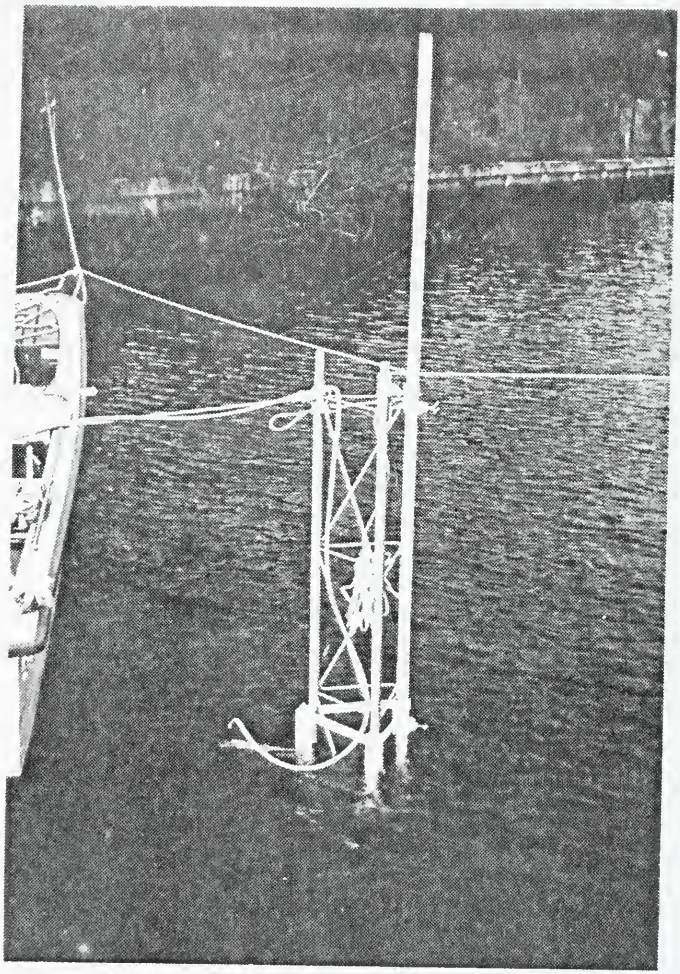


Figure 6.3 - Velocity Meter Tower Set-Up in Canal.

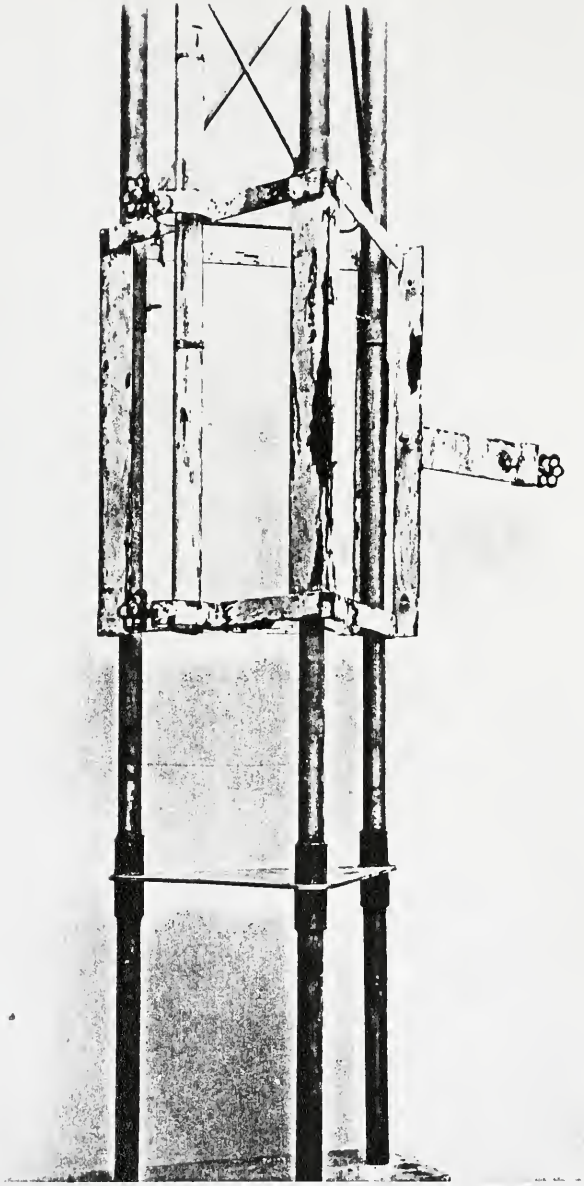


Figure 6.4 - Lower Part of Velocity Tower Showing Carriage With Probe Holder and Adjustable Legs, Designed and Built by Snyder Oceanography Services.

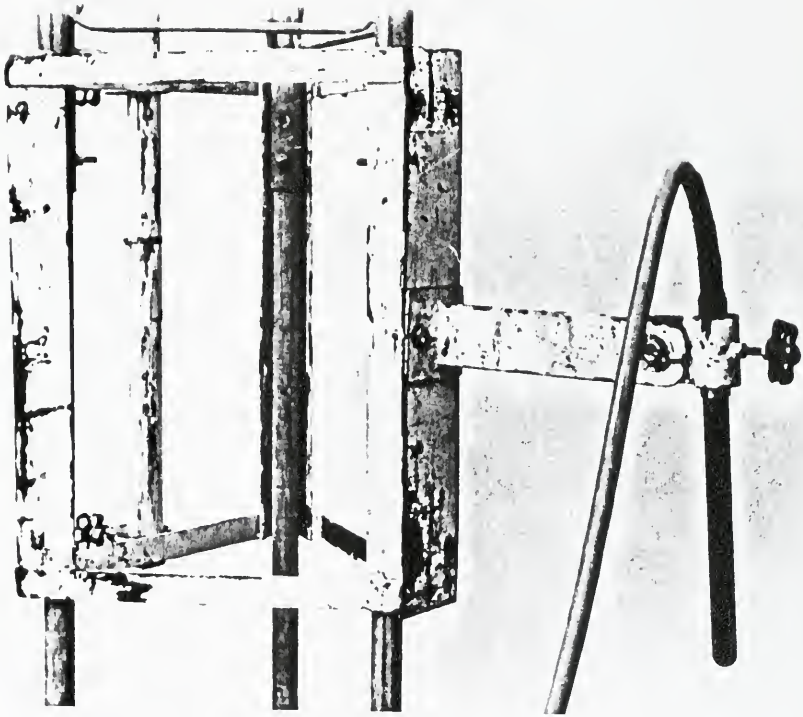


Figure 6.5 - Detail of Probe Holder Carriage on Velocity Meter Tower, With Probe Installed.

Figure 6.6 - Three Velocity Meter Towers Installed in the Loxahatchee North Canal.

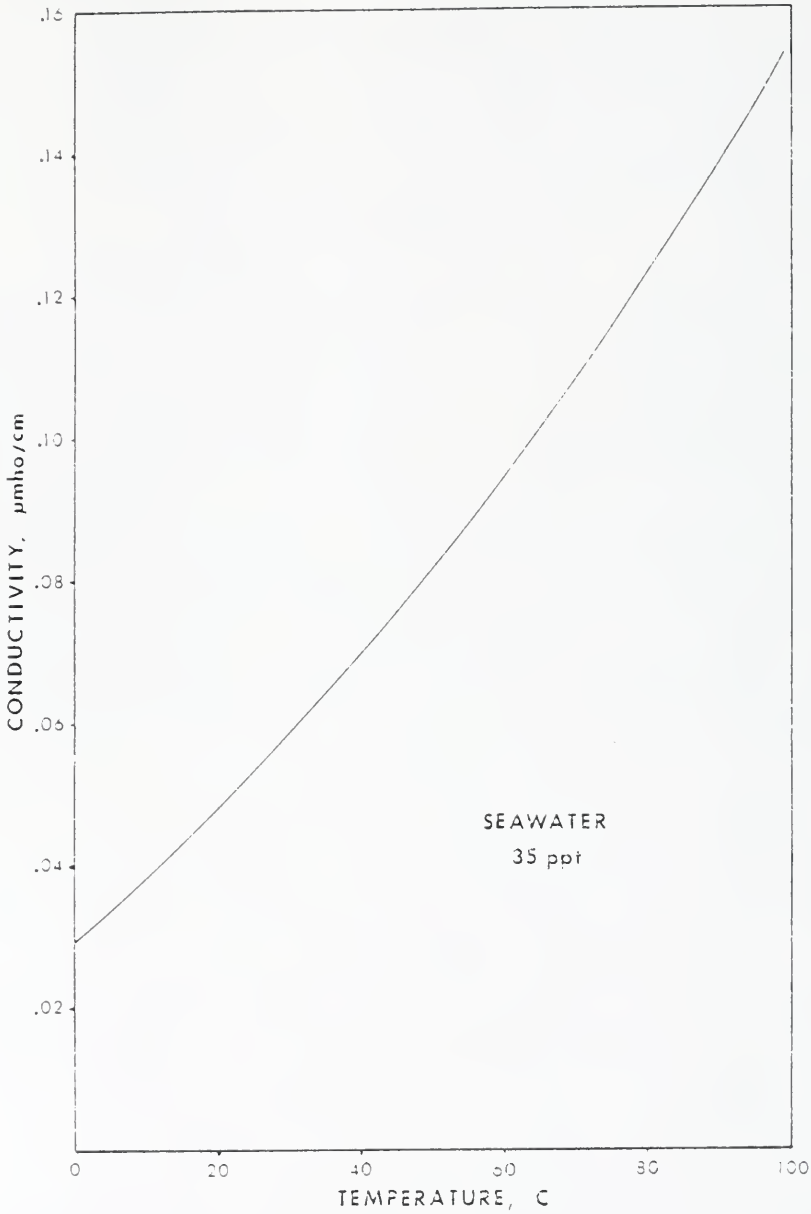


Figure 6.7 - Electrical Conductivity of Seawater as a Function of Temperature.

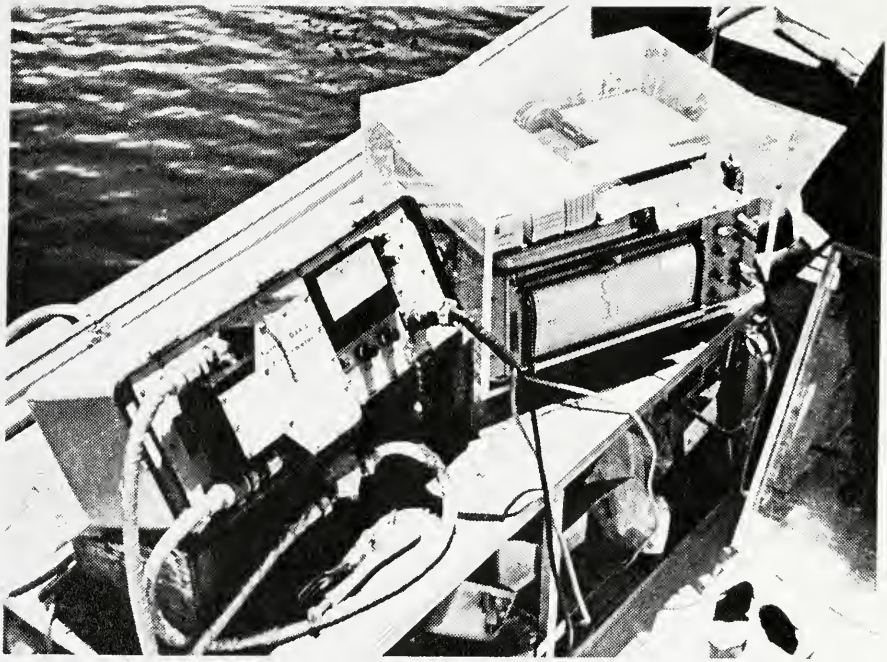


Figure 6.8 - Fluorometer with Continuous Sampling Arrangement and Strip Chart Recorder on Work Boat.

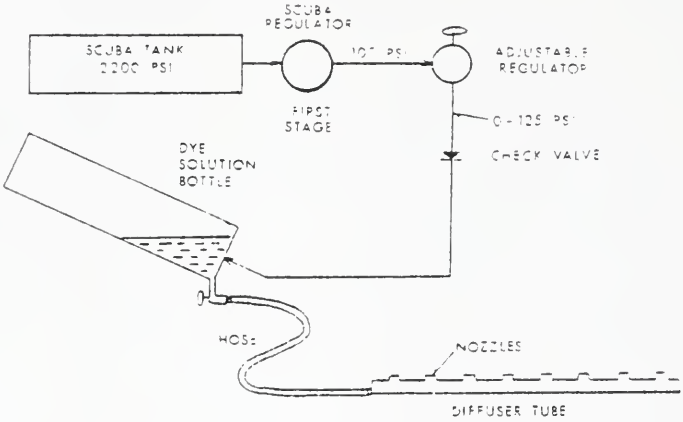


Figure 6.9 - Pressurized Dye Injection Device Constructed by Snyder Oceanography Services.

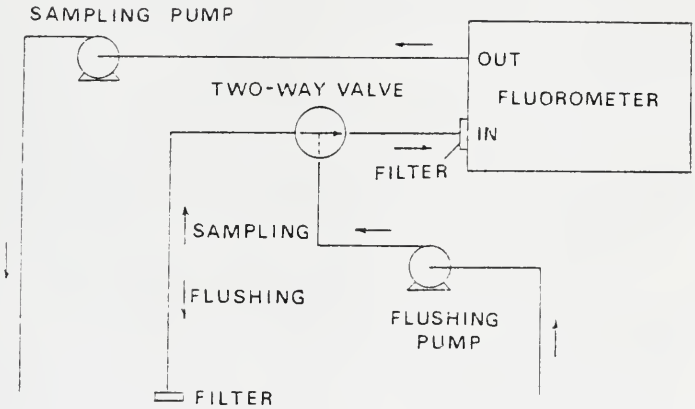


Figure 6.10 - Diagram of Continuous Flow Dye Sampling System.

Figure 6.11 - Use of Short Length of PVC Pipe for Sampling Water-Tracing Dye from Moving Boat at 3 ft depth.

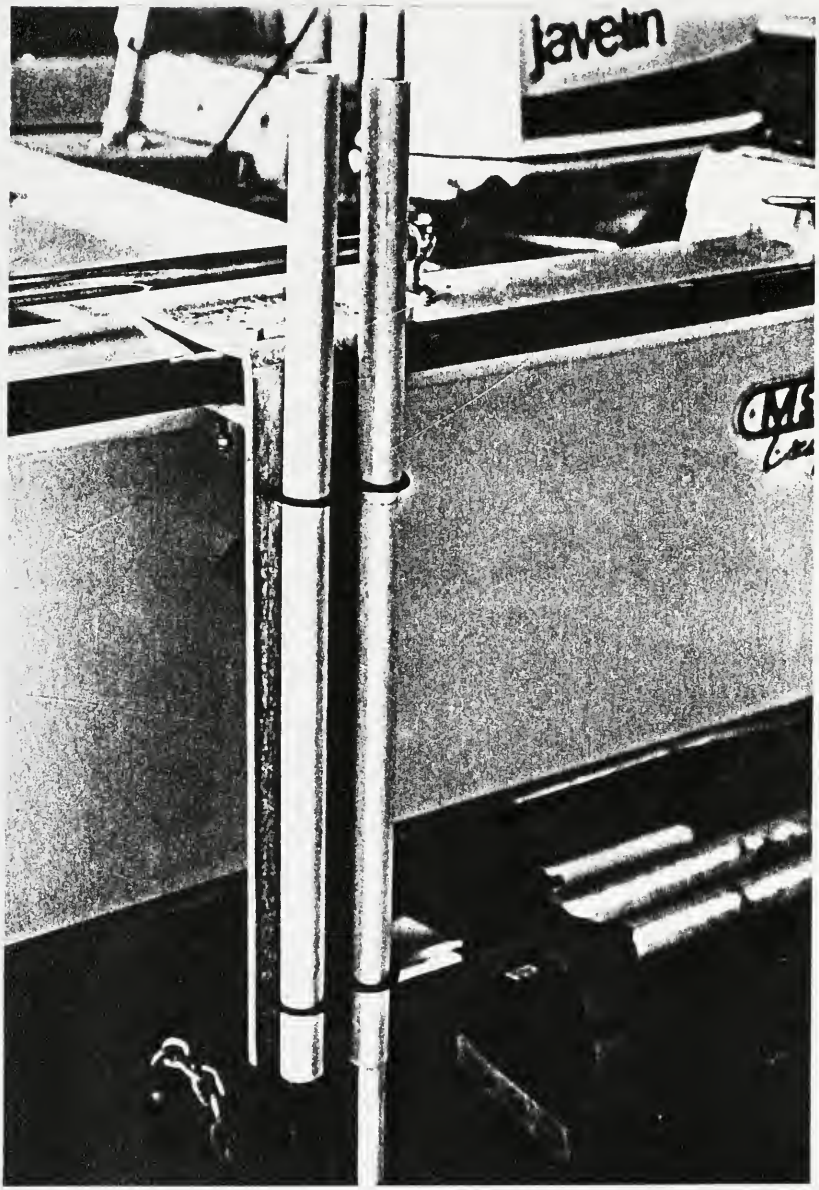


Figure 6.12 - Gunwale Support for 3 and 6 ft Water Sampling Tubes.

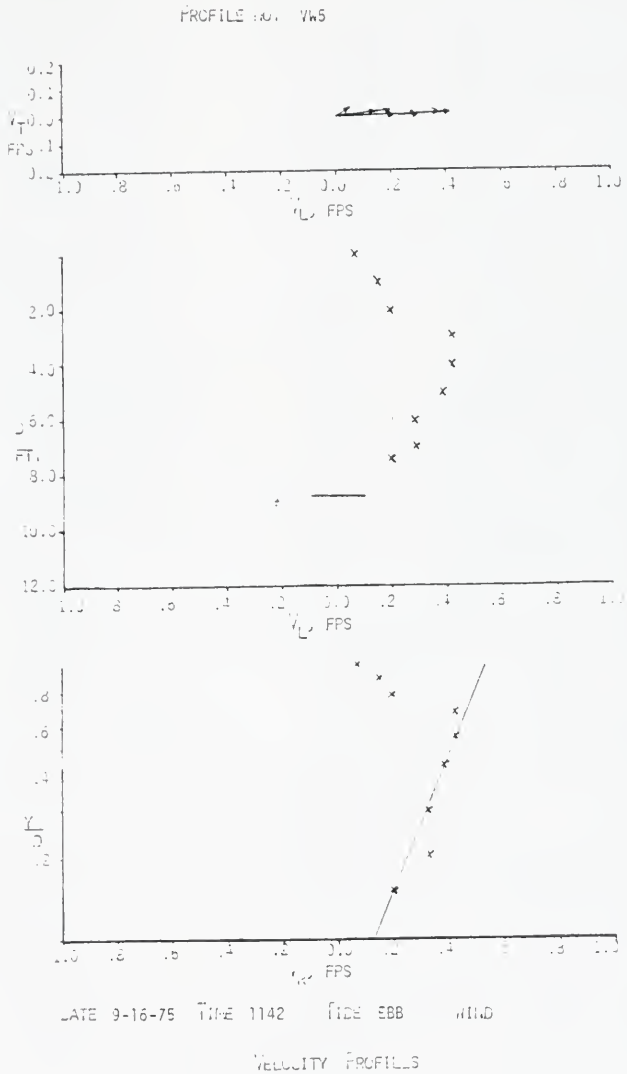


Figure 6.13 - Example of Plotted Velocity Components from Measurements by Electromagnetic Current Meter.

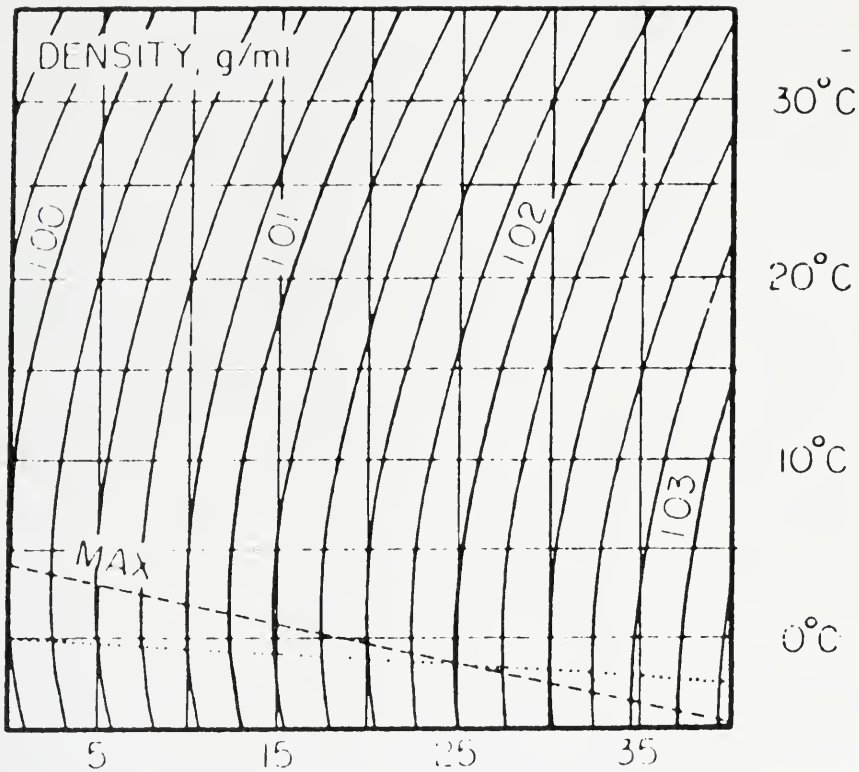


Figure 6.14 - Relationship of Density of Seawater (g/ml) to Salinity (ppt) and temperature (°C). (Source: Chow, 1964, p. 2-5).

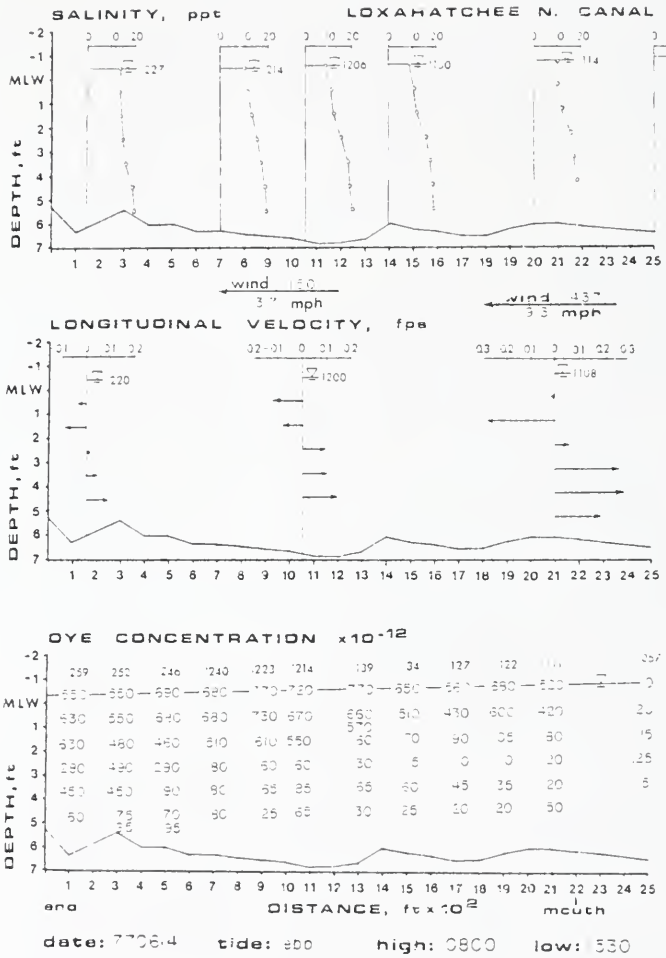


Figure 6.15 - Example of Plotted Salinity, Velocity, and Dye Concentration Profiles.

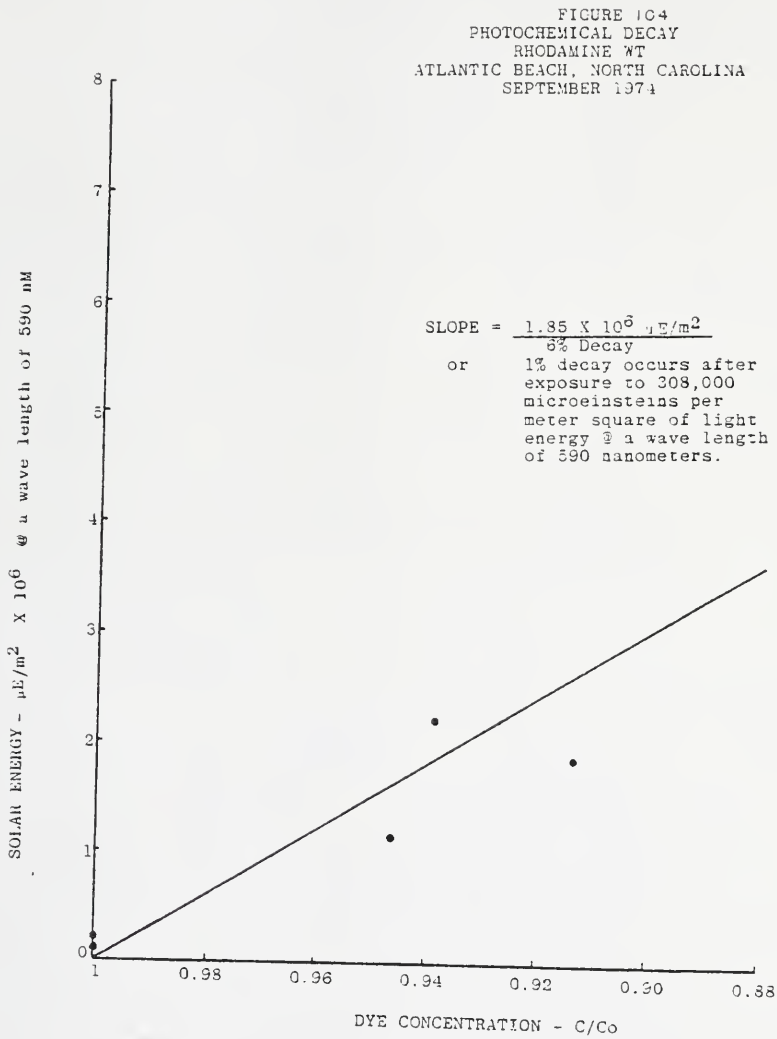


Figure 6.16 - Photochemical Decay Rhodamine WT Atlantic Beach, North Carolina September 1974 (Source: EPA, May 1975, p. 217).

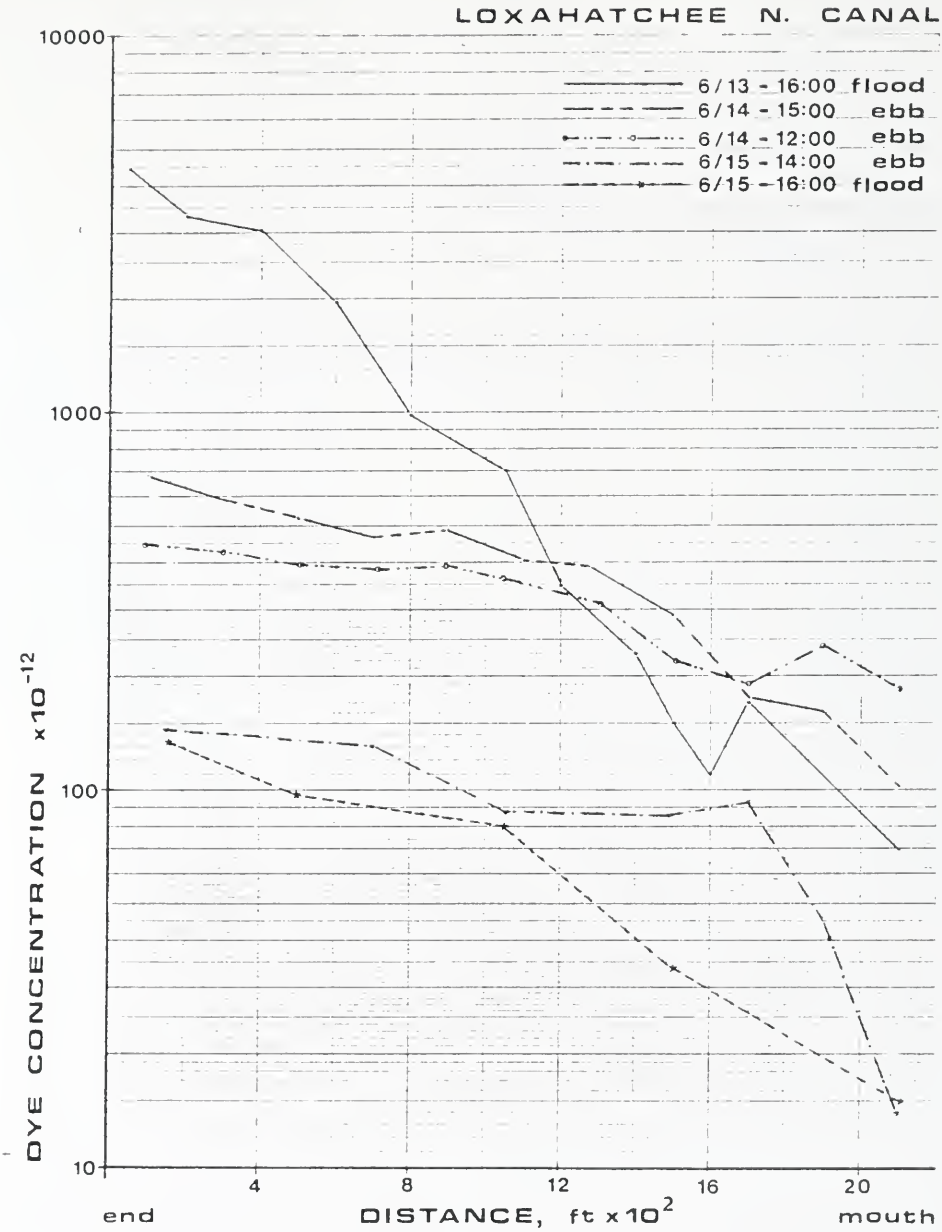


Figure 6.17 - Example of Vertically-Averaged Dye Concentration Profiles.

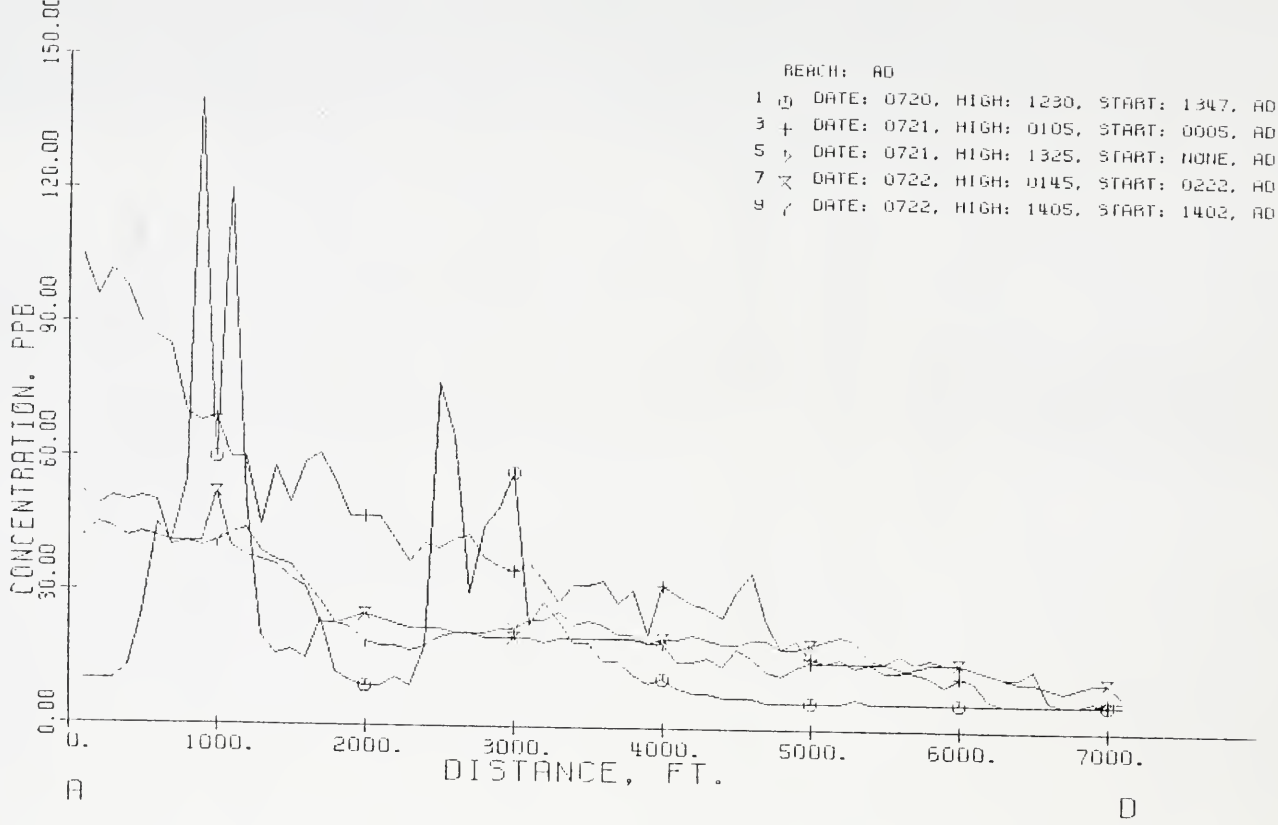


Figure 6.18 - Example of Continuous Dye Concentration Profile.

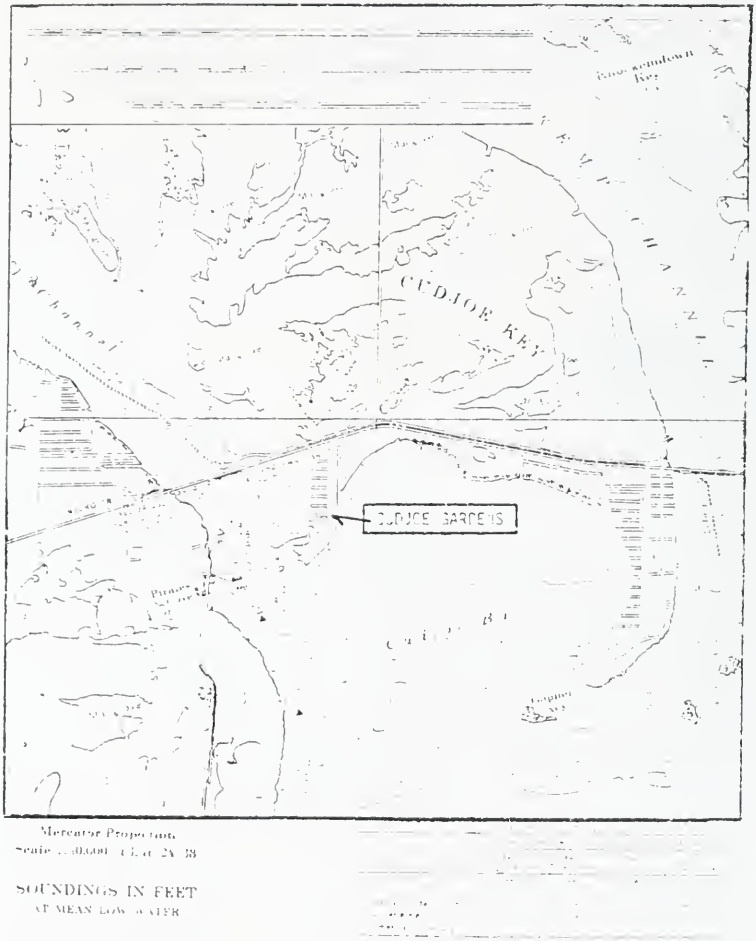


Figure 6.19 - Topographic (7 1/2 min. quadrangle) Map for Cudjoe Gardens Canal System.

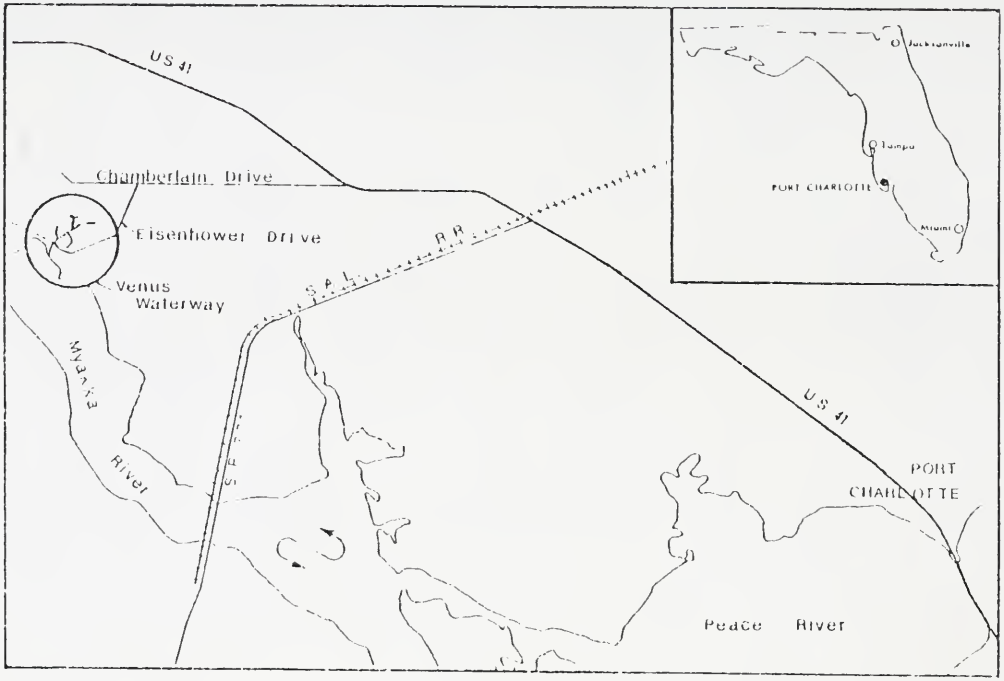


Figure 6.20 - Location Map for Venus Waterway Canal System.

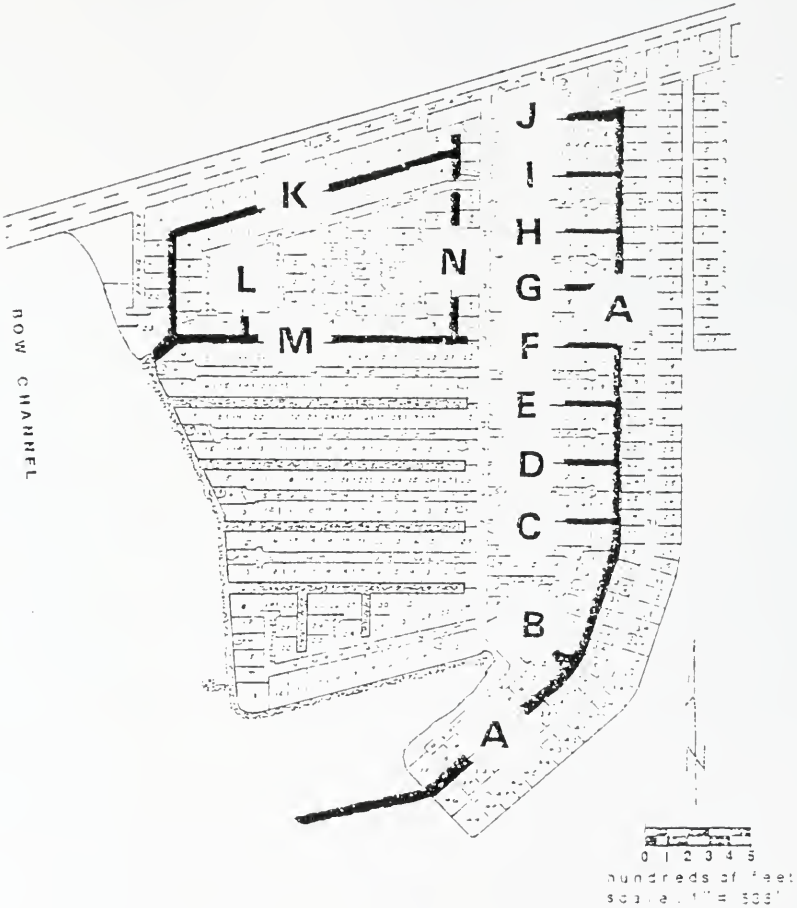


Figure 6.21 - Layout of Cudjoe Gardens Canal System, Cudjoe Key, Florida, with Canal Designation Letters Established for October 1974 Hydrographic Survey.

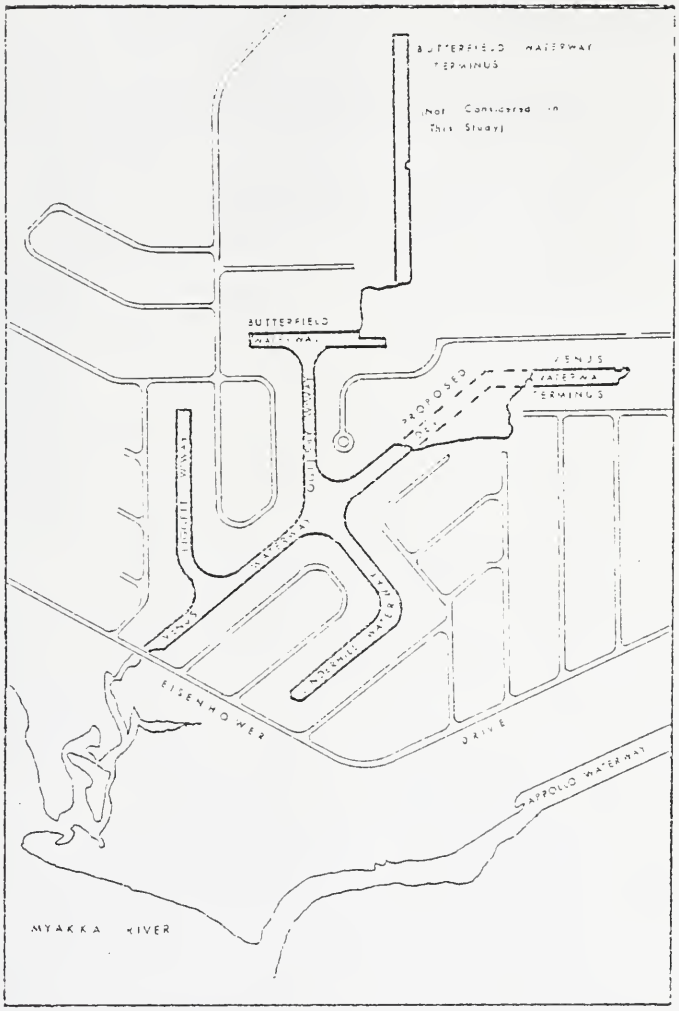


Figure 6.22 - Layout of Venus Waterway Canal System and Location of Venus Waterway Terminas.

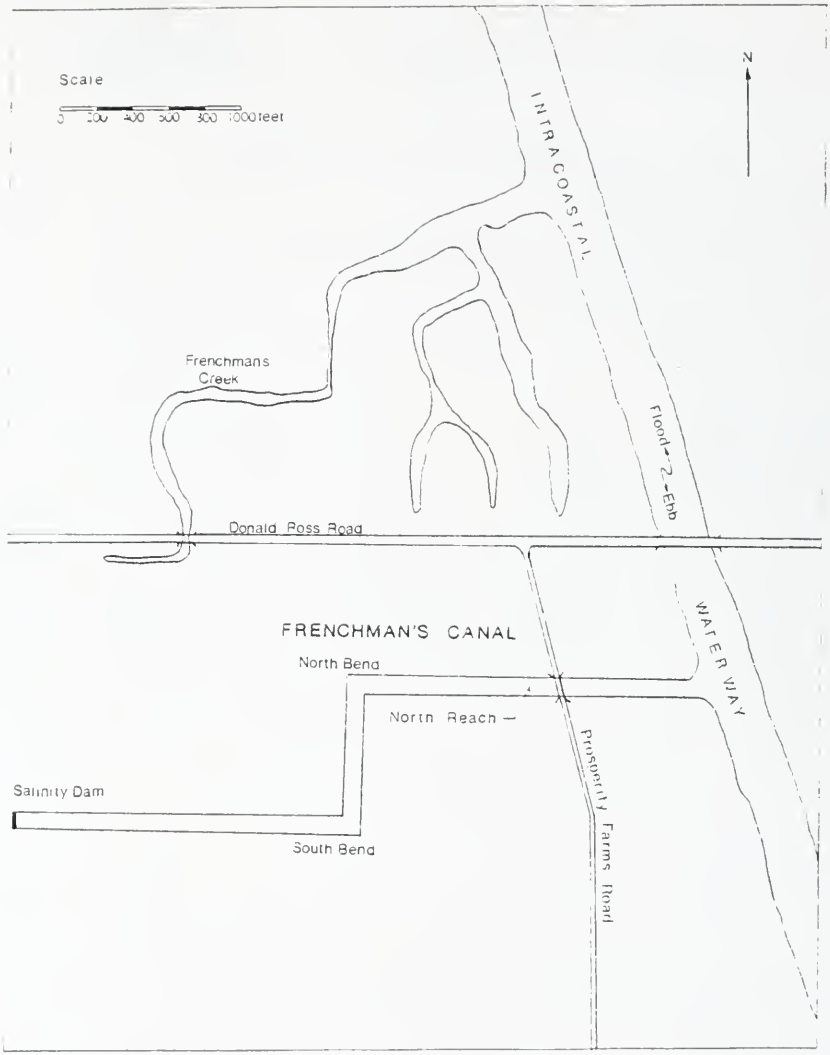


Figure 6.23 - Layout of Frenchman's Canal.

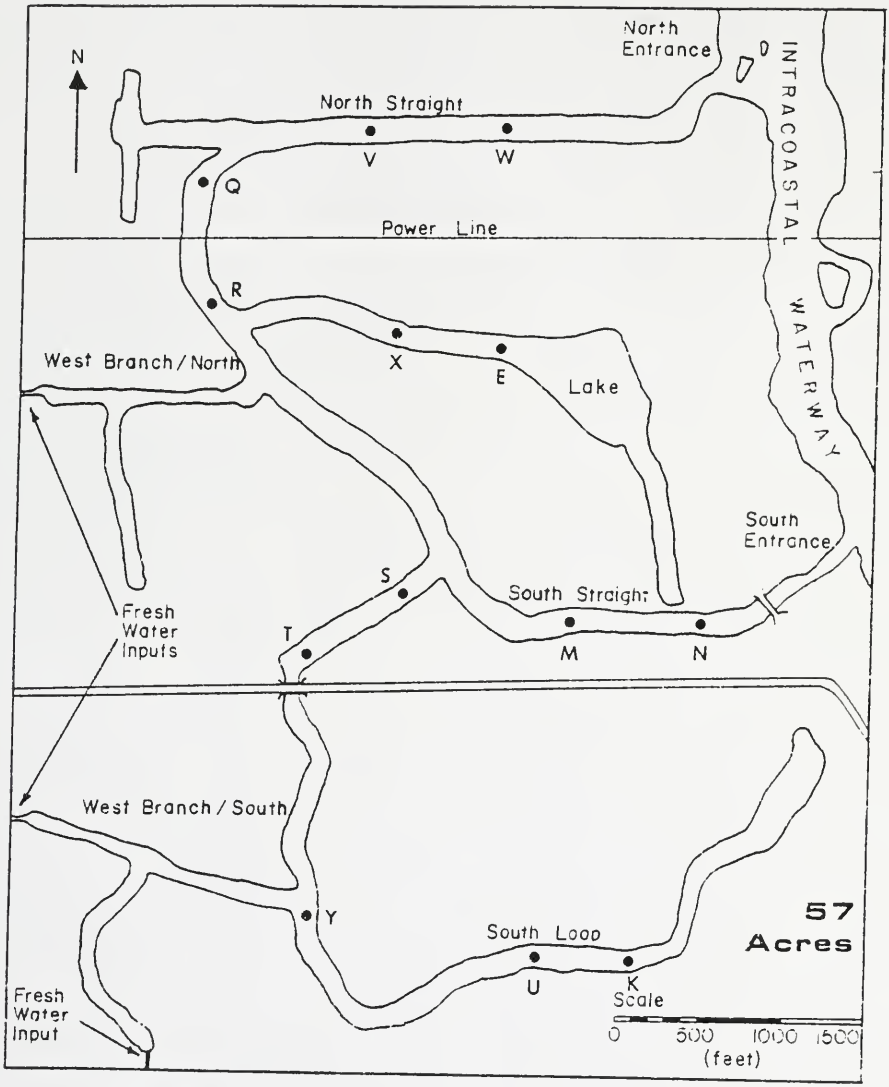


Figure 6.24 - Layout of 57-Acres Canal Network Showing Locations Used to Designate Reaches for 1975 Dispersion Studies.

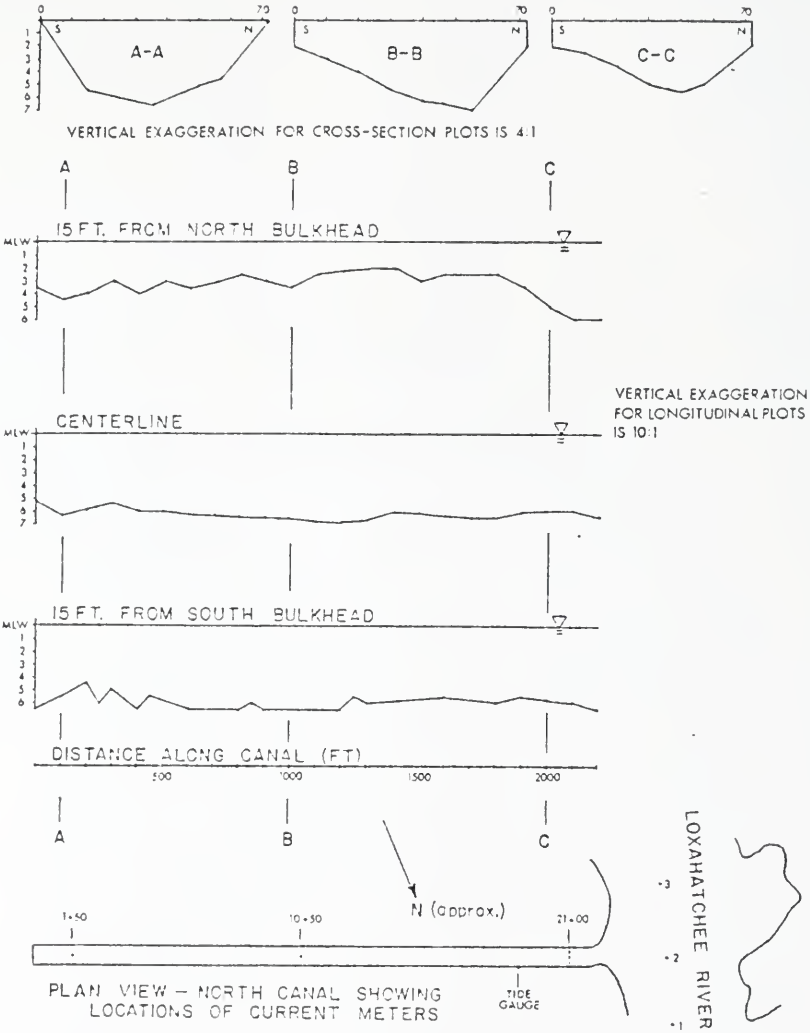


Figure 6.25 - Layout of Loxahatchee River North Canal, Showing Locations of Tide Gauge and Velocity Meters.

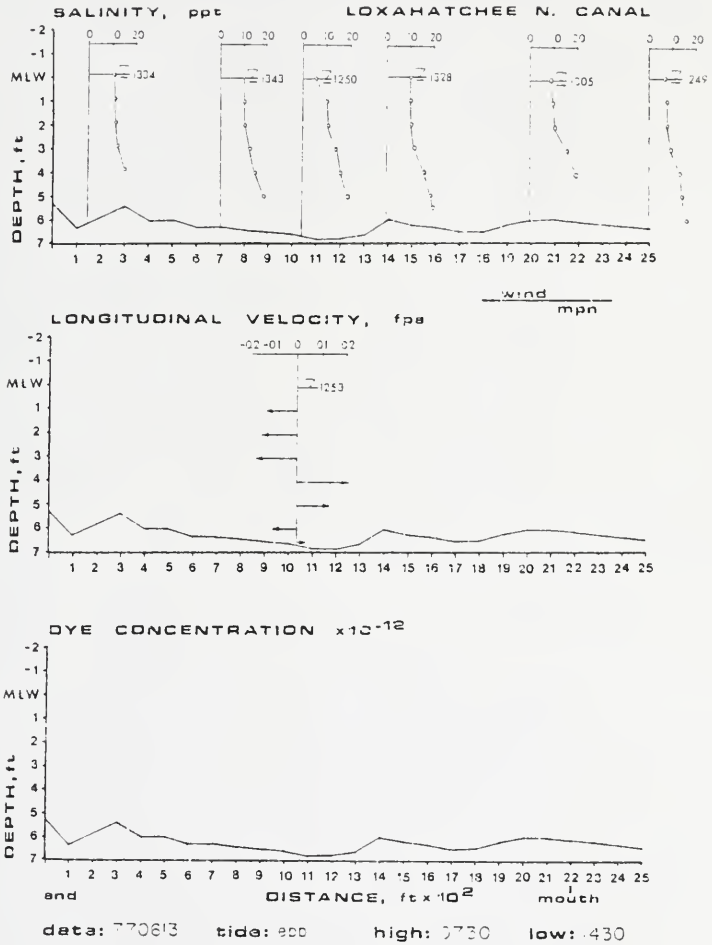


Figure 6.26 - Vertical Salinity Profiles, Loxahatchee North Canal, Showing Presence of Density Wedge Near the End of an Ebb Tide [Date: 770613; Time: 1249-1334].

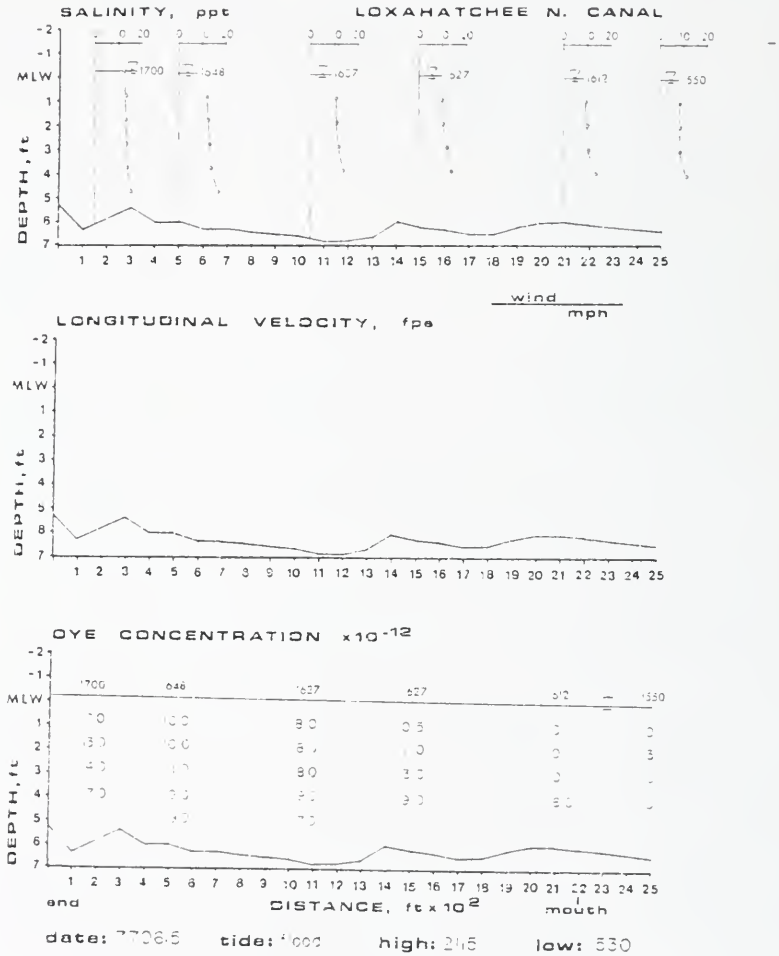


Figure 6.27 - Vertical Salinity Profiles, Loxahatchee North Canal, Showing the Beginning of Formation of a Density Wedge After Low-Tide [Date: 770615; Time: 1550-1700].

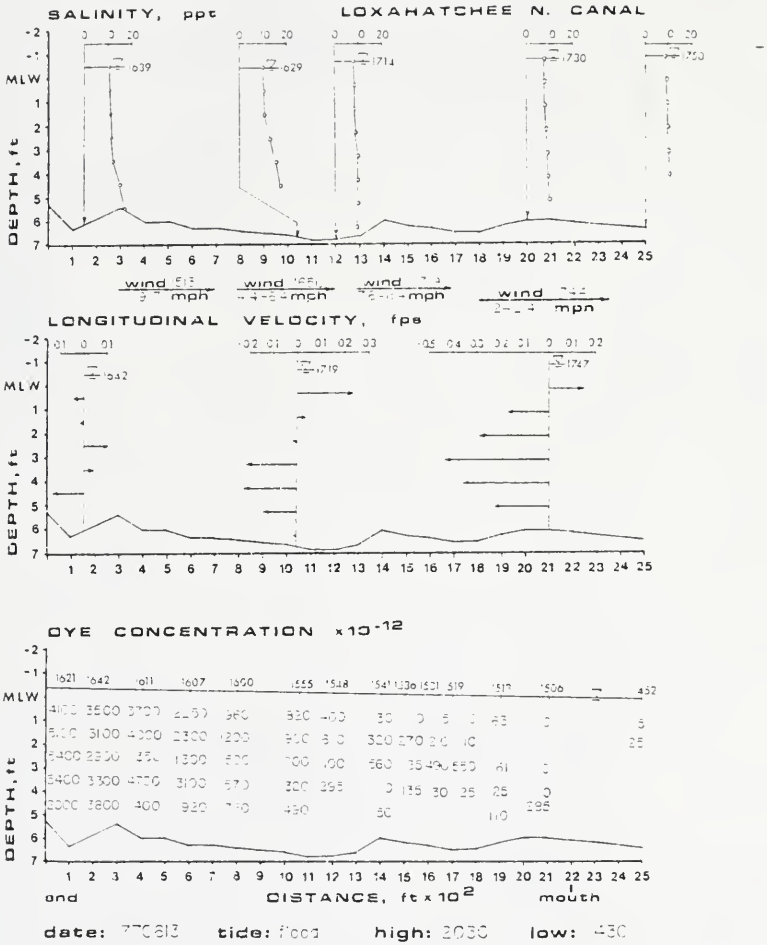


Figure 6.28 - Vertical Salinity Profiles, Loxahatchee North Canal, Showing a Remnant Salt Dome Near the Dead-End at Mid-Flood Tide [Date: 770613; Time: 1639-1750].

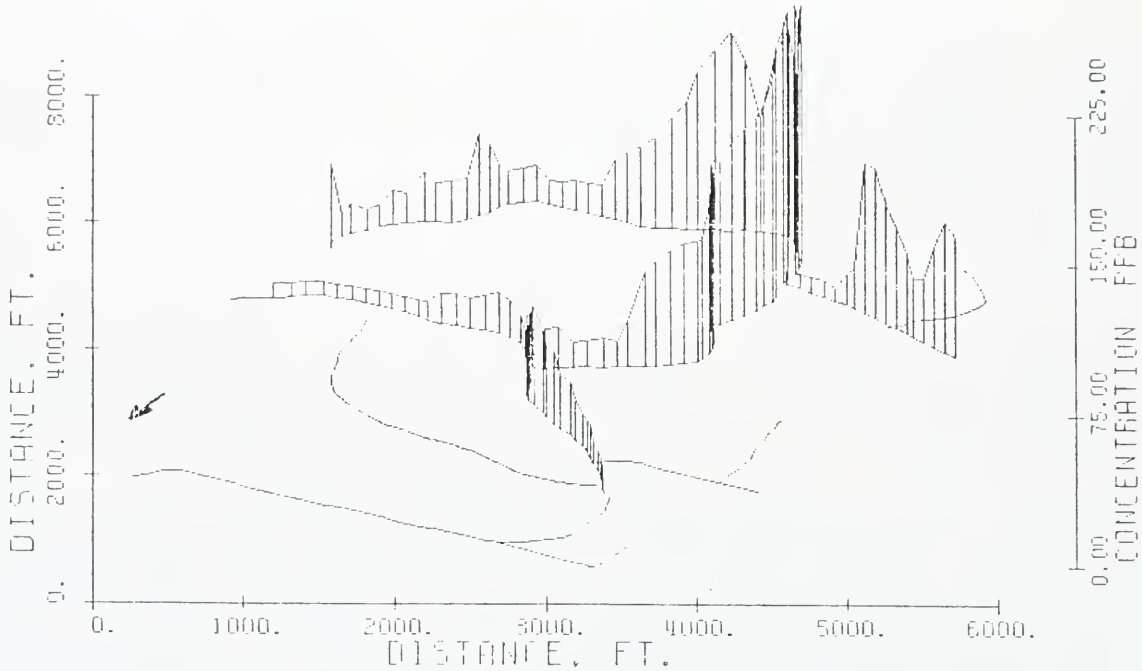


Figure 6.29 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, FL [Date: 770720; Time: 1935].

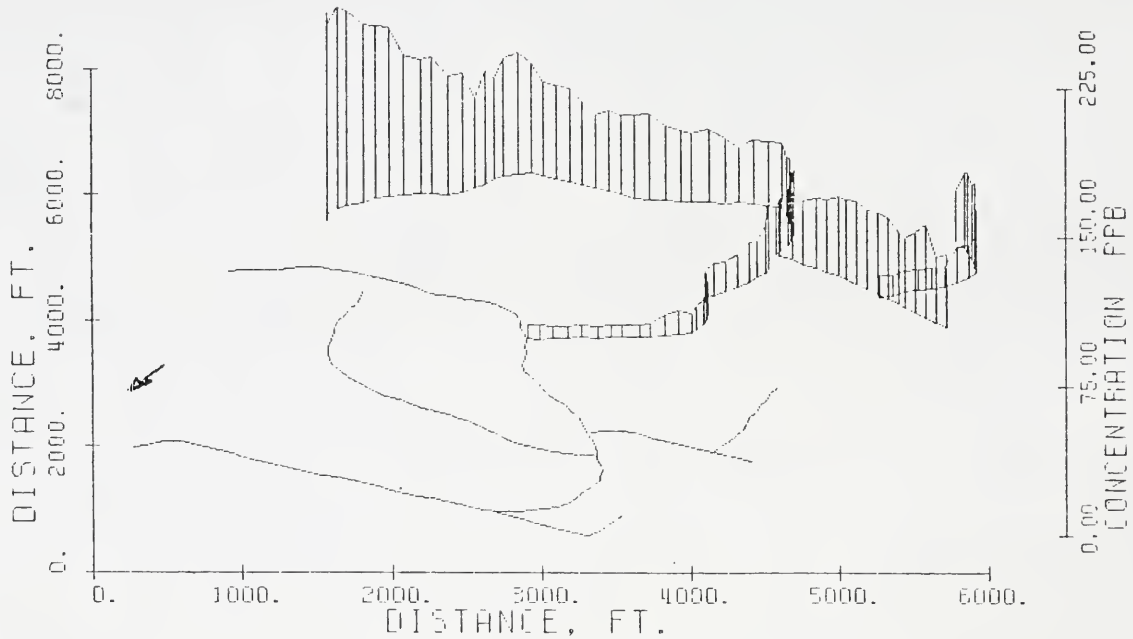


Figure 6.30 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, FL [Date: 770721; Time: 0105].

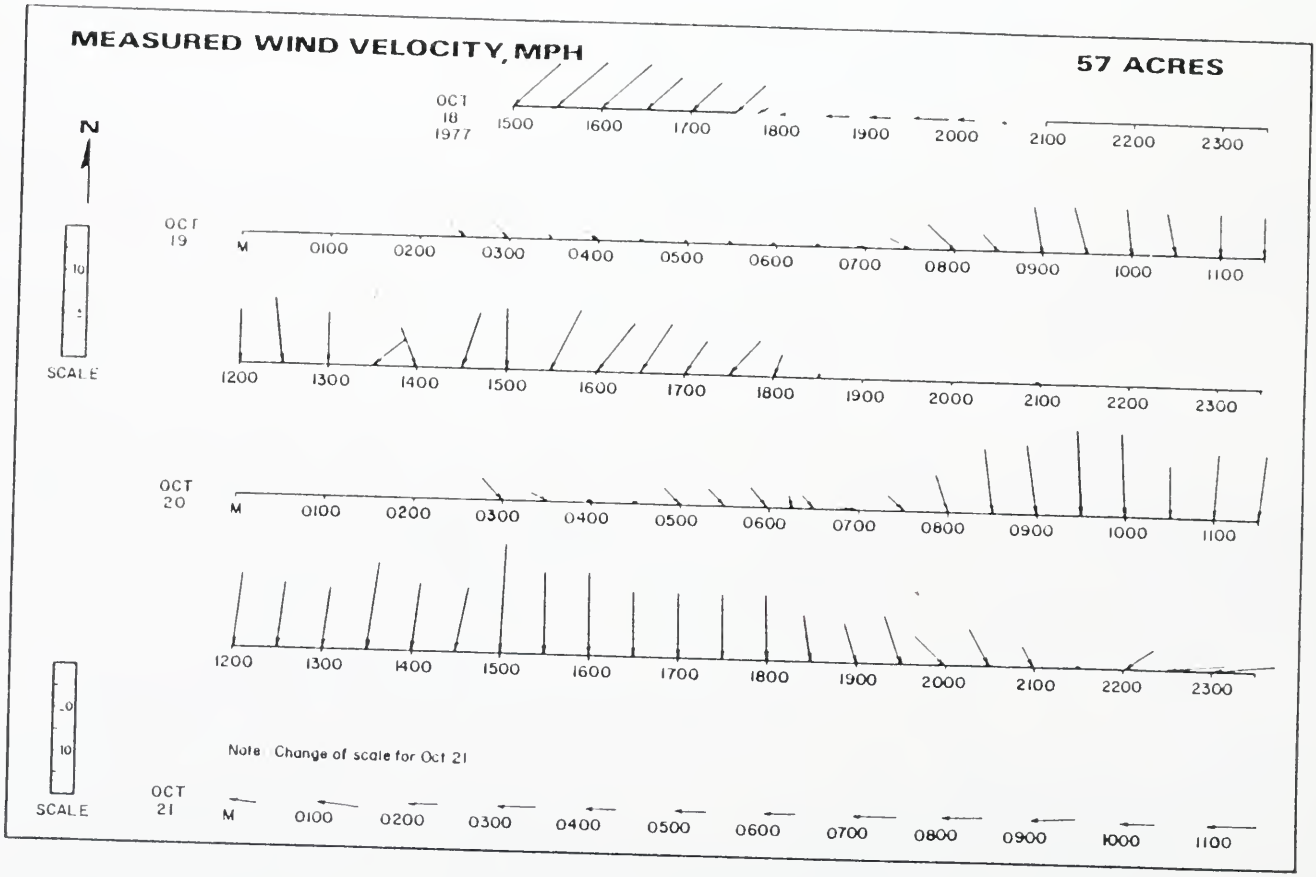


Figure 6.31 - Measured Wind Velocity, 57 Acres Canal System, October 1977.

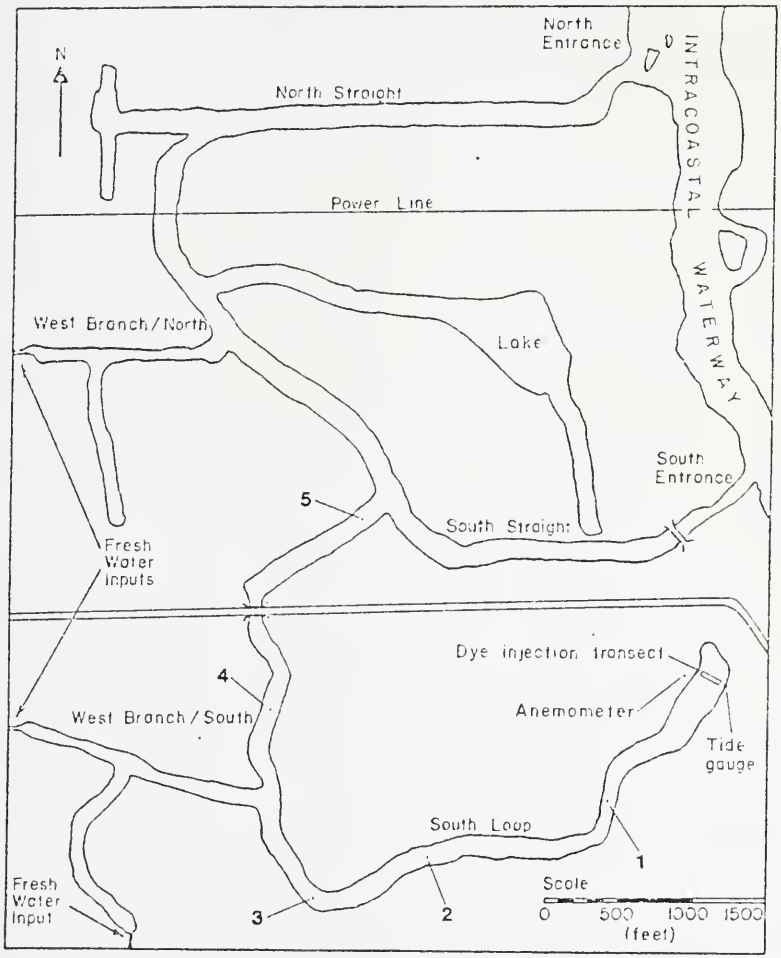


Figure 6.32 - 57 Acres Site Plan Showing Location of Electromagnetic Current Meters for October, 1977 Velocity, Salinity, and Water Temperature Measurements.

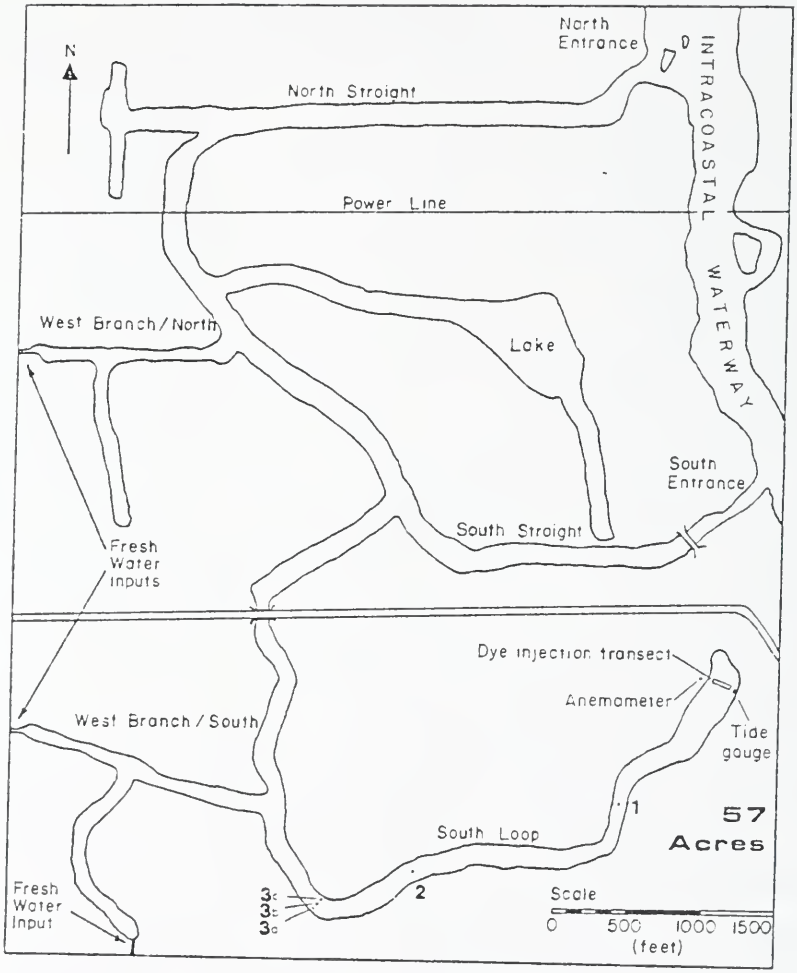


Figure 6.33 - 57 Acres Site Plan Showing Location of Electromagnetic Current Meters for October, 1977 Dye Dispersion Measurements.

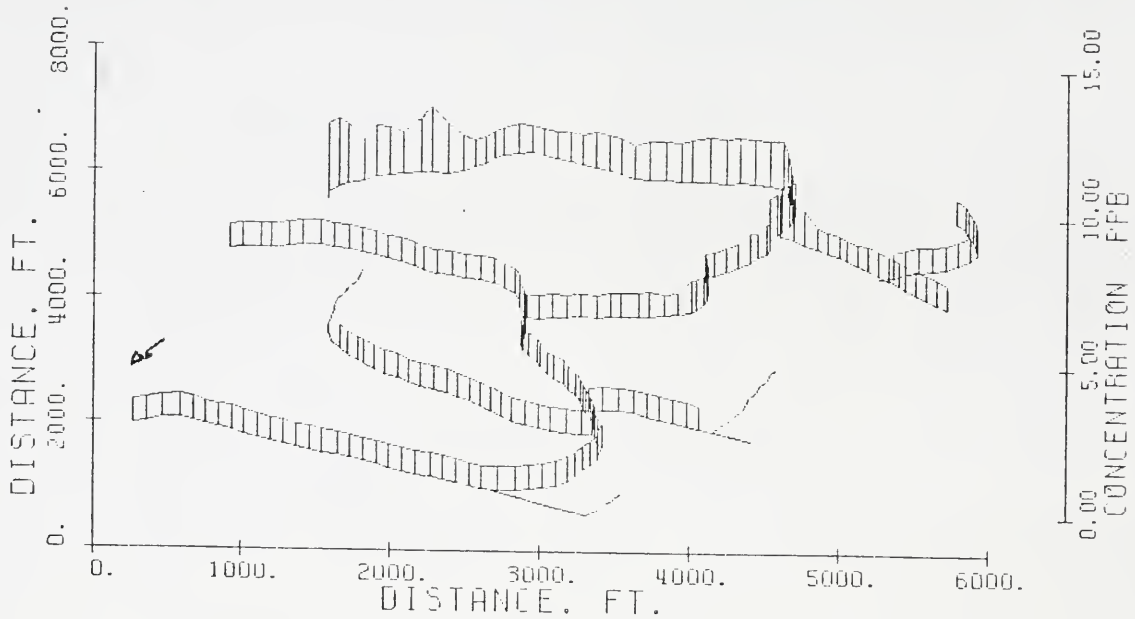


Figure 6.34 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, FL [Date: 771021; Time: 1045].

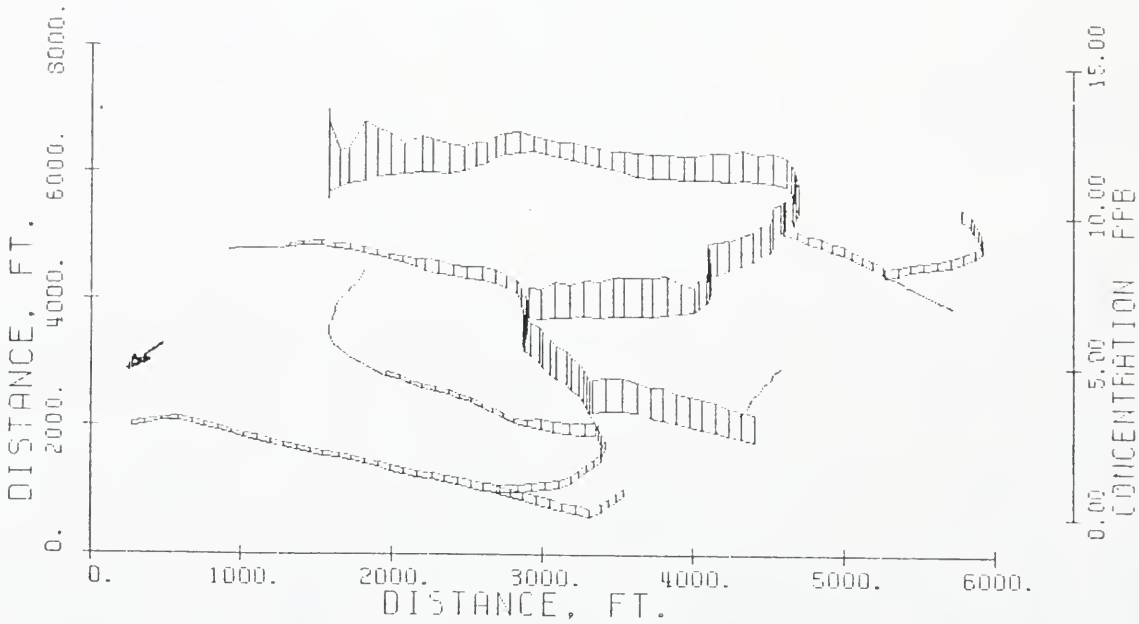


Figure 6.35 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, FL [Date: 771021; Time: 1045].

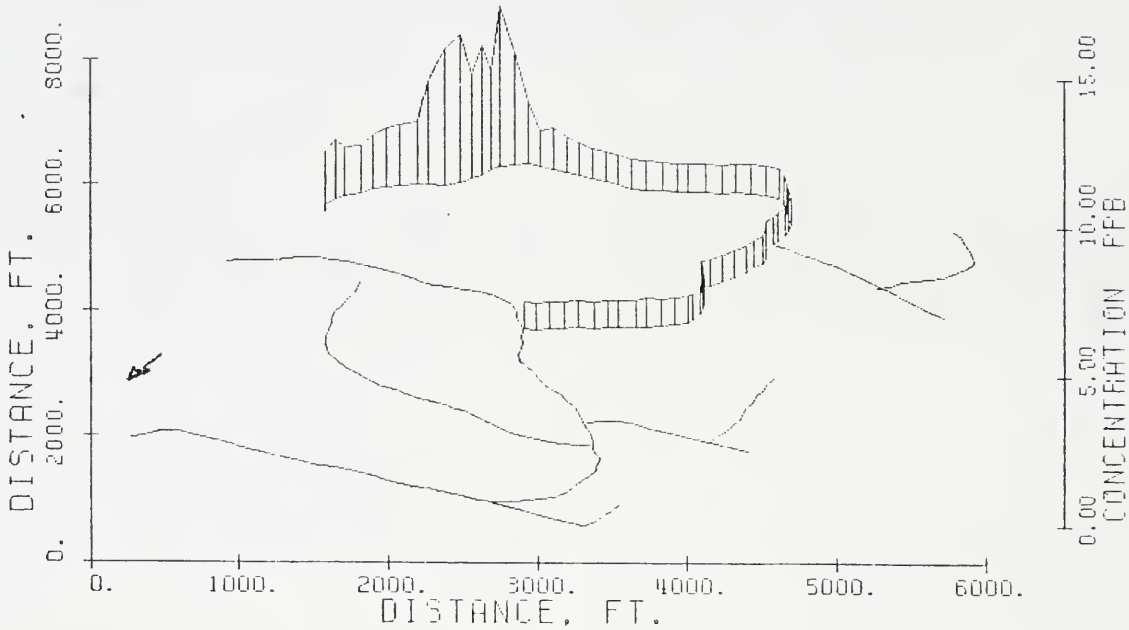


Figure 6.36 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, FL [Date: 771020; Time: 0330].

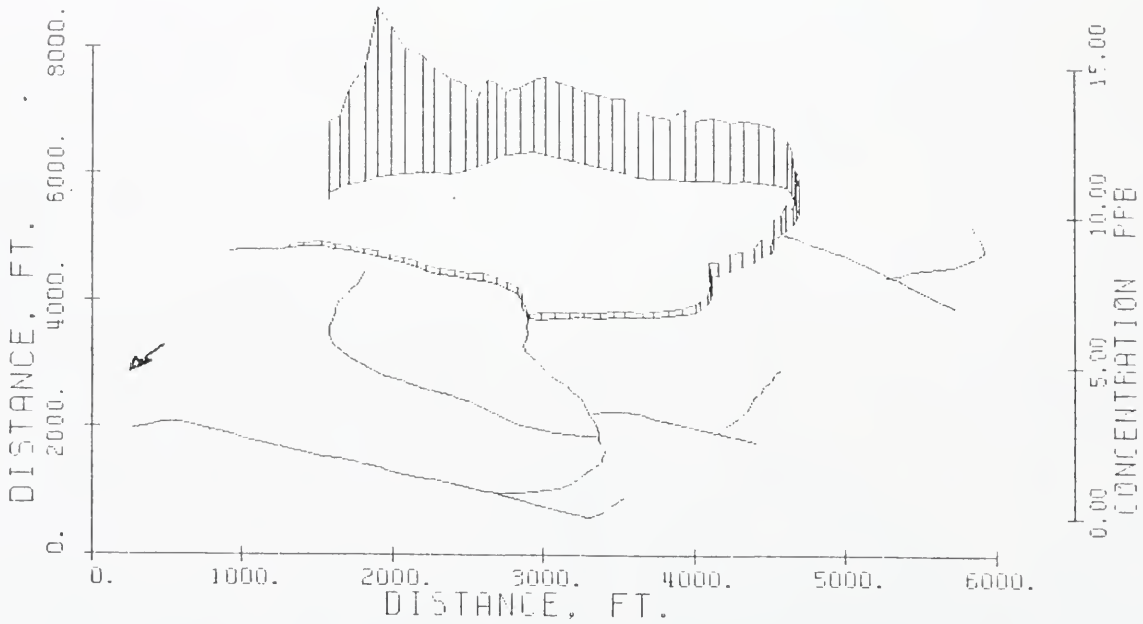


Figure 6.37 - Measured Dye Concentrations at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, FL [Date: 771020; Time: 0950].

CHAPTER 7

THE NUMERICAL CANAL NETWORK MODEL

The capability for analysis of the pollutant transport characteristics of a particular canal network is based on a numerical model developed for the Florida Sea Grant canal design research project by R. Walton [1978]. This three-dimensional model evolved from a series of one-dimensional models, which proved to be unable to satisfactorily simulate observed conditions in tidal canal networks, but which nonetheless contributed in various ways to the ultimate development of the three-dimensional canal model. In this chapter the basic concepts and applicability of these models will be reviewed, particularly in context with the results of field measurements. This review will include the verification of the three-dimensional model with the results of the field surveys described in Chapter 6.

This chapter is a condensation of Chapters 3 through 9, written by R. Walton in the final report to Florida Sea Grant [Morris, Walton and Christensen, 1978]. It is included in order to provide continuity to the research reported herein, and all of the essential information from Chapters 3 through 9 is contained here. Many portions have been taken verbatim from Walton's text, in order to preserve the accuracy of his presentation, while others have been freely paraphrased. It should be understood that the author, while involved in the development of some of the concepts in the modeling approach (particularly the hybrid computer

model), has not contributed any substantial original work to the development of this numerical model.

7.1 Development of the One-Dimensional Models

The formulation and testing of one-dimensional models provided several basic concepts which were applicable to the multi-dimensional model. These include a simple expression for tidal velocities and the longitudinal dispersion coefficient, methods for handling boundary conditions, and an appreciation for numerical dispersion, conservative and non-conservative properties, transportiveness, and stability. These features are summarized in this section.

7.1.1 Definitions of Canal Network Geometry

The simplest canal configuration is a straight, prismatic *reach* with trapezoidal cross-section and uniform depth. Many residential canals in Florida are of this form, as for example those along the Intracoastal Waterway (ICW) in Pompano Beach, Florida. Several such reaches or branches may be joined at a *junction*, the *branch* canals often joining a *main* canal at right angles (Figure 7.1). The main canal, in turn, is connected hydraulically at one end to the *communicating waters*, often called the *receiving waterbody*, which may be the ICW, the ocean, or a bay or estuary. A canal *network* is considered to branch upwards from the *tidal entrance(s)*, each successive level of branches being denoted as *secondary*, tertiary, and higher branches, which ultimately lead to *dead-ends*. A dead-end of a reach may be either an impervious boundary, a salinity control structure, or a point of entry of a tributary or overland flow at which further excursion of the tide is effectively prohibited.

A further geometric complexity found in some canal networks is a *loop*, which is defined as "that part of a canal network in which a closed continuous line can be drawn along the longitudinal centerlines of component reaches" [Walton, in Morris, Walton and Christensen, 1978, p. 141]. A canal network with two or more tidal entrances is considered to be composed of a single loop, which in certain cases can be simulated with this model. Loops may also occur between reaches, but these cannot be accomodated in this model.

In a network with two tidal entrances it may be postulated that there is some area approximately equidistant from the two entrances at which, on a flood tide, the flow from the two entrances will meet. At this location it can be further assumed that the velocities will be very small and there will be little interaction. Such an area may be defined as a *null point*, which in a symmetric network is at the geometric located center. If the tides are in phase, or approximately in phase, at the two entrances and the null point is not located within a second- or higher-order reach, then a network with a loop (i.e., two tidal entrances) can be simulated with this model.

In this project, the positive x-axis for each component reach is considered to be defined from the point of the reach which is hydraulically located farther from the tidal entrance. Flow in the positive x-direction is called *downstream* flow, while in the negative x-direction it is called *upstream* flow. Facing downstream the *left bank* is defined as the bank to the left of the centerline, and the *right bank* is defined as the bank to the right of the centerline.

At a junction, which can be the meeting point of from two to four distinct reaches, the *downstream reach* for that junction is the reach

hydraulically closest to the tidal entrance. The *upstream reach* is the remaining reach if there are only two, or the reach which is a more continuous extension of the downstream reach if there are three, or the reach in the middle if there are four reaches all together at the junction. Then it follows that, looking downstream, the *left branch* refers to the reach joining the junction from the left and the *right branch* is the branch joining from the right.

The geometry of a model canal network is constructed of reaches and junctions. It is assumed that each reach is trapezoidal in cross-section, with bottom width b, left and right bank inverse side slopes s_L and s_R, cross-sectional area A, and depth d. Then,

$$A = d(b + sd) \tag{7.1}$$

It was convenient, in setting up the computational scheme, to refer to each dead-end as junction number 1, the "interior" junctions (all junctions which are not dead-ends or tidal entrances) as junctions number 2 through (NJUNC + 1), where NJUNC is the number of interior junctions, and to number the tidal entrances consecutively from (NJUNC + 2) upwards. When a "lake," or large waterbody, is included to increase the tidal prism in portions of a network, it is represented by a junction numbered sequentially after the last interior junction but before the tidal entrance junction(s). The lake-type junction is best formulated with a longitudinal dimension (DXJN) the same as in other junctions in the network, and a lateral dimension (DYJN) of the size required to provide the desired area.

The transport, i.e., the convection and spreading, of a substance introduced into a time-varying flow is described by a convective-diffusion or a convective-dispersion equation. Such an equation is an

expression for the concentration, c (x,y,z,t), as a function of the mean velocity components u, v, and w, and diffusion coefficients E_x, E_y, and E_z, in the three coordinate directions x, y, and z, respectively, and the inflow or outflow of the substance at sources and sinks. The hydrodynamic equations provide a description of the velocity components as functions of location and time, and the mass transport equation for a substance is obtained from an expression for the conservation of mass of the substance. These equations will be described in the following sections.

7.1.2 One-Dimensional Hydrodynamics

The principal phenomena affecting the flow in canal networks are the astronomical tide, wind-induced circulation, density- and temperature-induced currents, secondary currents, and lateral inflow. All of these components, with the exception of lateral inflow, produce circulation patterns which have significant components in at least two coordinate directions and thus cannot be realistically modeled with a one-dimensional model. Compensating for this effect by introducing coefficients is rarely satisfactory, because by doing so the model will then be limited to the conditions existing at the time the data base for the coefficients was obtained, and still will be unrealistic.

Most hydrodynamic models are based on the de Saint Venant equations of continuity and the momentum equation (frequently called the dynamic equation), which are commonly given in texts on fluid dynamics. Measurements of hydrodynamic variables in Florida's coastal finger canal networks indicate, however, that it is usually not necessary to include the momentum equation. A typical Floridian canal network can be subdivided into reaches less than 10,000 feet long, separated by junctions

with negligible bed slopes. The astronomical tidal range is about 2 to 4 ft on the Atlantic Coast, and 2 to 3 ft on the Gulf Coast. This small tidal range results in very small tidal velocities in canals, usually less than 0.5 fps and often less than 0.1 fps in magnitude. Maximum measured water surface slopes are of the order of 10^{-5} to 10^{-6}.

The observation of almost negligible water surface slopes led Christensen [1975] to suggest that the acceleration terms in the momentum equation would also be negligible, and that a good approximation for the tidal velocity could be derived by assuming that the water surface is always horizontal throughout the canal network. Then the continuity equation is sufficient to describe the velocity field [Walton, 1976a], given by

$$u_i(x,t) = \frac{1}{A_i} [\sum_j^{Nu} \int_0^{L_j} q_{I_j} \, dx + \int_0^{x_i} q_{I_i} \, dx] - \frac{A_{ws_i}}{A_i} \frac{dd}{dt} \qquad (7.2)$$

where

u_i = cross-sectionally averaged longitudinal tidal velocity at section i, (L/T)

$A_i(t)$ = cross-sectional area at section i, (L^2)

Nu = number of upstream reaches

L_j = length of reach j, (L)

q_I = lateral inflow per unit length of reach, (L^2/T)

x_i = longitudinal distance from upstream section of reach i, (L)

$A_{ws}(t)$ = canal network water surface area upstream of section i, (L^2)

d(t) = tidal depth, (L)

The horizontal water surface assumption was tested by developing a simple model based on the continuity equation

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} - q_I = 0 \qquad (7.3)$$

where

Q = discharge, (L^3/T)

q_I = lateral inflow per unit length of reach, (L^2/T)

t = time, (T)

x = longitudinal displacement, (L)

and comparing the computed depths, water surface slopes, and velocities from this model with the computed results using Harleman and Lee's [1969] model based on both the continuity and the momentum equations. For these comparisons many of the canal parameters (given in Table 7.1) were varied, including in particular the canal length. In all cases the depths and velocities agreed within 2 percent, as shown by the comparison in Table 7.2. It was decided that this was sufficiently accurate, considering the order of magnitude of errors inherent in numerical modeling and in the measurement of data in the field.

7.1.3 One-Dimensional Mass-Transport

The basic equation used for modeling the transport of a substance in a flow is a partial differential equation which may be derived from a general expression for the conservation of mass of the substance. The conservation of mass equation can be extended to turbulent flow by applying Reynold's rules of averaging. The resulting turbulent mass flux terms can be related to concentration gradients through an analogy with Fick's first law, resulting in a set of turbulent diffusion coefficients to replace the turbulent mass flux terms e_t. Finally, recognizing that for flows of practical interest the turbulent diffusion coefficient is very much greater than the molecular diffusion coefficients still remaining in the turbulent form of the equation, a substitution of the Fickian form provides the three-dimensional

mass-transport equation for turbulent flow in a homogeneous (constant density) waterbody [Harleman, 1966, pp. 576-578; Pritchard, 1971, p. 16]

$$\frac{\partial}{\partial t}(Ac) + \frac{\partial}{\partial x}(Auc) + \frac{\partial}{\partial y}(Avc) + \frac{\partial}{\partial z}(Awc) = \frac{\partial}{\partial x}(Ae_{t_x}\frac{\partial c}{\partial x})$$

$$+ \frac{\partial}{\partial y}(Ae_{t_y}\frac{\partial c}{\partial y}) + \frac{\partial}{\partial z}(Ae_{t_z}\frac{\partial c}{\partial z}) + Ar_p \qquad (7.4)$$

where

c = concentration averaged over a turbulent time period long enough for $\bar{c}' \to 0$ but not long enough to significantly damp variation of c with respect to t, (dimensionless)

= turbulent (or eddy) diffusivity coefficients in the coordinate directions, i, (L^2/T)

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

x,y,z = coordinate directions, (L)

u,v,w = velocity components in x,y,z directions respectively, (L/T)

Note that the cross-sectional area can also be a function of location and time, in this form of the equation.

By cross-sectionally averaging the three-dimensional mass-transport equation, and again using the Fickian analogy, a form of the one-dimensional mass-transport equation can be derived [Harleman, 1971, pp. 37-38]. This equation contains both a turbulent diffusion coefficient, e_t, which is associated with the *temporal* deviations of velocity component u and concentration c, and the x-direction dispersion coefficient E_x, which is associated with the *spatial* deviations of u and c. Recognizing [Taylor, 1954] that the dispersion coefficient E_x is an order of magnitude greater than the turbulent diffusion coefficient e_t, a longitudinal dispersion coefficient may be defined

$$E_\ell = E_x + e_t \qquad (7.5)$$

The rate of production of the substance, r_p, may be expressed in a more convenient form by assuming the substance to be passive and con-

servative and that there are no sinks for the flow except at the tidal entrance. Then the rate of production is equivalent to the lateral inflow, q_I. Designating the concentration of the substance in the lateral inflow as c_I, the rate of production is

$$r_p = q_I c_I / A \tag{7.6}$$

and the common form of the one-dimensional mass-transport equation (also called the convective-dispersion equation) is

$$\frac{\partial}{\partial t}(Ac) + \frac{\partial}{\partial x}(Auc) = \frac{\partial}{\partial x}(AE_\ell \frac{\partial c}{\partial x}) + q_I c_I \tag{7.7}$$

The longitudinal dispersion coefficient is often expressed in the form first given by Taylor [1954] and Elder [1959].

$$E_\ell = KRu^* \tag{7.8}$$

where

K = dimensionless dispersion coefficient

R = hydraulic radius, (L)

u^* = bed shear velocity, (L/T)

The dimensionless dispersion coefficient K is a function of the geometry of the canal network, the bottom roughness, radii of bends, wall Reynolds' number, and other factors which affect local mixing. From Elder's and Taylor's experiments the value of K ranged from 5.9 to 20.1, but other researchers have since obtained values from less than 10 to the order of several hundreds in natural waterbodies [Fischer, 1967b, p. 188].

The Hydraulic Laboratory has measured many vertical velocity profiles in Floridian canals, some when the effects of wind and density gradients have been negligible. A logarithmic velocity profile provides a unique expression between the spatial mean velocity, u, and the bed shear velocity, u^* [Nikuradse, 1933]

$$u = 2.5\ u^{*}\ \ell n(10.9\ d/k) \tag{7.9}$$

where

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

Substituting Equation (7.9) into (7.8) results in the expression

$$E_{\ell} = \frac{0.4\ KRu}{\ell n(10.9\ d/k)} \tag{7.10}$$

Thus the no-wind, spatial mean velocity (Equation 7.2) and the longitudinal dispersion coefficient (Equation 7.10) can be expressed analytically as functions of x and τ.

7.1.4 Boundary Conditions

The one-dimensional mass-transport equation (Equation 7.7) is a second-order parabolic equation. Since it is second-order in the spatial variable x, and first-order in the time variable t, two spatial boundary conditions and a set of initial conditions are required for a numerical solution.

The first boundary condition was applied at the dead-end of a reach. In the beginning a zero-flux condition, $\partial c/\partial x = 0$, was used until it was realized that simple numerical approximations, such as a forward-difference operator

$$(c_1 - c_0)/\Delta x = 0 \tag{7.11}$$

where

c_o = concentration at dead-end, (dimensionless)

c_1 = concentration at the adjacent section, (dimensionless)

implied that the gradient of the concentration profile in the dead-end segment was zero, which led to erroneous results. Higher-order operators were considered, but these utilize information for locations

farther away from the dead-end than is realistic for the problem. It was finally decided to represent the concentration at the dead-end as the linear extrapolation of the concentrations at the two section adjacent to the dead-end section

$$c_0 = 2c_1 - c_2 \tag{7.12}$$

The second boundary condition was applied at the tidal entrance. A dual condition was established: during the ebb tide the concentration at the entrance was calculated using a backward difference operator, while on flood tide a first-order decay was used to describe the decrease in concentration from the low tide value as a function of time. Thus, on ebb tide, the concentration at the tidal entrance, C_{TE}^{n+1}, is given by

$$\frac{c_{TE}^{n+1} - c_{TE}^{n}}{\Delta t} = \frac{u_{TE}^{n}\, c_{TE}^{n} - u_{TE-1}^{n}\, c_{TE-1}^{n}}{\Delta x} \tag{7.13}$$

where

Δt = time increment, (T)

Δx = spatial increment, (L)

n = time level

while on the flood tide

$$c_{TE} = c_{RW} + (c_{LT} - c_{RW}) \exp(-3t'/\tau) \tag{7.14}$$

where

c_{RW} = background concentration of the receiving waterbody, (dimensionless)

c_{LT} = concentration at previous low tide, (dimensionless)

t' = time since low tide, (T)

τ = time decay coefficient, (1/T)

This latter boundary condition causes the tidal entrance concentration to decrease to within 2 percent of the background concentration, c_{RW}, after τ units of time following low tide. Subsequent tests showed that the first-order decay condition gave quite reasonable approximations, and furthermore, variability tests with values of τ showed that changes in τ do not substantially change the resulting concentration profiles in the canal (see section 8.5).

The initial conditions for any simulation are specified in terms of the concentration at time t=0. In typical simulations this concentration is either the background amount, c_{RW}, or some value specified for each section, each reach, or the entire canal network.

7.1.5 Numerical Methods

An analytic solution for the one-dimensional partial differential mass-transport equation (Equation 7.7) with time and space-dependent velocity and dispersion coefficients has not yet been found. Accordingly, numerical approximations are used to solve this equation for concentration c(x,t). A variety of numerical techniques can be used, each having limitations of varying degrees of importance. The development and testing of each of these techniques is covered by R. Walton in Morris, Walton, and Christensen [1978], and will be summarized in this section.

7.1.5.1 Finite-Difference and Finite-Element Methods

Finite-difference techniques are the most often used methods for numerically solving differential equations. The one-dimensional mass-transport equation (Equation 7.7) consists of convective terms on the left-hand side and a dispersive term and a lateral inflow term on the

right-hand side. Approximations to these differential terms are formed by considering the Taylor series expansion of a function about a known value and ignoring higher-order terms. However, when the higher-order terms in the expansion of the convective part of the equation are ignored, and the approximations are substituted for the differential terms, a new term is formed which is dispersive in nature. The effect of this dispersive term, which is called numerical dispersion or pseudo-dispersion, is added to the desired effect of the dispersion term and the solution of the equation either spreads the concentration profiles too rapidly or causes instabilities in the results, depending on the relative sizes of the dispersion terms and their sign. If the desired dispersion coefficient is large the effect of numerical dispersion may be relatively unimportant, but in the case of Floridian canals the measured longitudinal dispersion coefficient was found to be quite small, on the order of 5 ft^2/sec, and the numerical dispersion in finite-difference approximations can easily be of this order of magnitude.

Numerical dispersion can be reduced by reducing the spatial increment, Δx, in the model. However, this results in more computational steps and can increase computer costs substantially, especially since the time increment Δt usually must also be decreased to meet stability criteria. Tests using reduced spacial and temporal increments were conducted for a single straight canal reach, and it was concluded that the sizes of increments which would reduce numerical dispersion to acceptable values precluded the use of a finite-difference technique for canal networks.

Similar tests with a finite-element model showed similar dependencies on small spatial and temporal increments. In addition, since the advantage of flexibility in the specification of boundary geometry was not required for the canal network models, the finite-element method was also discarded.

7.1.5.2 Hybrid Computer Approach

A hybrid computer is an analog computer which is controlled by a digital computer. The analog part consists of summing amplifiers, multipliers, integrators, and coefficient potention meters which can be remotely changed by the digital computer, and associated electronic components. The characteristics of a system (in this case, the hydrodynamics and concentration distributions in a canal) can be expressed mathematically, and in turn the mathematical expressions can be represented by an analog circuit composed of these electronic components. The digital part of the hybrid computer is used to set those analog variables which have been implemented with potentiometers, to set the initial conditions, and, if desired, to collect and analyze results. A simulation of the system can then be conducted on the analog computer by varying boundary conditions and plotting the concentration values as a function of time, as in an all-digital computer simulation.

The principal advantage of an analog or hybrid computer is that the simulation can be run with continuous time instead of with incremental time steps. Theoretically, with a central-difference operator for the spatial differences in the one-dimensional mass-transport equation the numerical dispersion is only dependent on the time increment. Another advantage, therefore, is that numerical dispersion can be eliminated, with a proper formulation of the model on an analog or hybrid computer.

A third advantage is that simulations can be run at an even faster speed on an analog computer than on a large, all-digital computer such as the Amdahl 470 used for the remainder of the project, at substantially less cost per run.

In order to investigate the possible use of a hybrid computer for simulating mass transport in a canal network, a feasibility study was initiated with the Hybrid Computation Laboratory at Martin-Marietta Aerospace in Orlando [Morris, Walton, and Christensen, 1977a; Sorondo and Baldwin, 1977]. Martin-Marietta was selected because the company has one of the largest hybrid computer installations in the country. Four different finite-difference formulations were implemented and could be selected by the digital program. In addition, for verification purposes the Martin-Marietta team programmed an equivalent all-digital finite-difference model for their digital computer. The test canal used for these simulations was a trapezoidal, constant mean depth, prismatic channel, with variable bottom width, side slope, mean depth, roughness, dimensionless dispersion coefficient, tidal amplitude, tidal entrance decay coefficient, and spatial increment. The number of sections in the canal was fixed at five (concentration was calculated at six points, including a point in the receiving water body).

The advantages of some aspects of the model were immediately apparent. Simulations over hundreds of tidal cycles could be run in a matter of seconds, and continuous analog plots of concentration vs. time at each of the sections immediately gave a realistic impression of the flushing activity in the canal. Many simulations with varying parameter values could be quickly run to check the variability of the canal. Similar plots of results from Martin-Marietta's all-digital model assured

convenient comparisons between different runs.

The hybrid model also had a number of disadvantages. Perhaps most significant was the limitation imposed by available hardware. It was found that about 20 canal sections could have been programmed on one console patch board, or 4 canal reaches with five sections in each. Although three separate consoles were available, it was impractical and too expensive to consider wiring all of these together and even then, there was not enough hardware available to consider two-dimensional canal networks with at least three vertical sections in each reach. In addition, once the patch-boards were set up, it became impractical to change the number of spatial increments or number of sections per reach, and it was obvious that it would not be convenient to change the geometry of a canal network often to provide a variety of test cases. It was primarily for these reasons that further consideration of the hybrid method was discontinued.

7.1.5.3 Second Upwind Differencing Method

The second upwind differencing method [Roache, 1972], also called the "donor cell" method by Gentry, Martin, and Daly [1966], is a finite-difference technique used by Lee [1977] to solve the one-dimensional mass-transport equation (Equation 7.7). This method uses a backward finite-difference expression for the convective term, written in terms of dischage rather than velocity, during the flood tide and a forward finite-difference term during the ebb tide. This method has several advantages over other finite-difference techniques. First, it is conservative, which means that the net accumulation of mass in a region equals the net flux across the boundaries of the region during the convective process. Second, it is transportive during the

convective phase, which means that "the effect of a perturbation in a transportive property is advected only in the direction of the velocity" [Roache, 1972, pp. 67-72]. "No stationary finite-difference formulation of the oscillatory convective process possesses this property" [Walton, in Morris, Walton, and Christensen, 1978, p. 165].

It was found initially that with this method, numerical dispersion was excessive. However, Lee [1977] extended an expression for the numerical dispersion in a first upward differencing scheme presented by Noh and Protter [1963] to the second upwind differencing method. In his development, he also extended the work of Bella and Dobbins [1968], who demonstrated that, once evaluated, the numerical dispersion could be subtracted from the natural dispersion by means of antidispersion flux terms. These terms are corrected or limited so that no antidispersion transfer of mass can increase the concentration at any grid point beyond the concentration value at neighboring grid points. The second upwind differencing method with flux-corrected transport and limited anti-dispersion is "essentially second-order accurate," and "even with the correction term remains both conservative in all steps and transportive in the convection step" [Walton, in Walton, Morris, and Christensen, 1978, p. 166]. This method proved to be relatively inexpensive to run on the Amdahl computer and gave very similar results to those obtained from the finite difference and hybrid models.

7.1.5.4 Method of Characteristics Techniques

Walton considered both movable- and fixed-grid versions of a method of characteristics model [Morris, Walton, and Christensen, 1978]. This method will not be discussed here since it has no direct bearing on the final version of the canal network model.

7.1.5.5 Method of Second Moments

The method of second moments, extended for the canal design project by Walton [Morris, Walton, and Christensen, 1978] is essentially an upwind finite-difference technique. However, in this method, instead of considering the magnitude of the concentration at a single point, the concentration distribution in each cell is represented by a rectangle having the same mass, same center of mass, and the same second moment squared. Thus, this rectangle may not fill the whole width of the cell, Δt, or its whole height, which depends on the instantaneous depth in the canal. In a finite-difference method, the convection of a square wave of concentration in a steady flow conserves only the zeroeth moment of the distribution (i.e., the mass), and the square distribution is rapidly spread and smoothed by numerical dispersion. In the method of second moments the first and second moment of the distribution are conserved as well, and inclusion of the second moment reproduces the initial square wave exactly during convection. However, numerical dispersion, caused by roundoff errors in the square root term, is still present in this technique.

For oscillatory flow, the technique is extended to account for changes in depth by expanding or contracting each cell according to the ratio of the new to the previous cross-sectional areas. Dispersion is modeled in each time step using the central finite-difference technique. Since the method of second moments is essentially an upwind-differencing method, it possesses the properties of conservativeness and transportiveness, as defined in section 7.1.5.3. The method was found to be the most accurate in convecting a square wave, which is one of the most severe tests that can be applied, and superior in its ability to

conserve mass. Also, it was observed to be more intuitively realistic than the second upwind differencing method, relatively easy to compute, and more economical than the other methods (except method of characteristics) considered. It was, therefore, determined to be the best model for extension to the three-dimensional case.

7.2 Development of the Three-Dimensional Model

The one-dimensional model of a tidal canal is useful for studying various numerical solution techniques, and for quantifying the effects of variations of some canal parameters on the resulting concentration profiles. It is unable, however, to simulate the transport of a conservative substance in canals under even very simplified circumstances, such as tidal action alone, because of the effect of the geometry on the flow and the way in which substances are introduced into the canals. The most important of these effects may be summarized as follows:

1. Flow caused by tide alone, in the absence of wind and density gradients, may be described by a logarithmic vertical velocity profile and an approximately parabolic lateral velocity profile. Thus, the mass-transport in a cross-section is not uniform.
2. Wind blowing over the water surface induces a surface flow in the direction of the wind and a flow reversal in the bottom layers. The momentum associated with these wind-induced currents causes them to lag the wind and integrate its directional effect.
3. Secondary-currents in the form of helical flows are caused by bends, and persist for some distance downstream of the bend.

4. Density gradients inhibit vertical mixing, but their effect on the flow field is difficult to model. Density wedges have been observed in some tidal canals, and the movement of the denser bottom water in the wedge into a canal on flood tide induces a freshwater counterflow in the surface layers. On ebb flow, a saltwater dome can be formed from the wedge.
5. A pollutant will be introduced at discrete locations along the canal, depending upon the type of source, and will therefore be subject to a "convective period" [Fischer, 1967] in which the spread of the pollutant is dominated by convection. Thus the substance, if introduced near the surface as in most field experiments, will tend to follow the surface layers and mix slowly into the lower layers. The distance associated with the convective period is typically on the order of the length of a canal reach (i.e., thousands of feet).

Thus, from the above, the difficulty in simulating the results of a tracer experiment with a one-dimensional model, which will be unable to incorporate any of the effects summarized above, is clear. If the tracer is introduced into the surface layers on an ebb tide, and a wind is blowing, it will be convected by the surface layer velocity and mixed slowly downward into the flow reversal zone, where it will be convected upstream. The combination of tidal flow and wind-induced vertical circulation will produce a tracer cloud with a sharp leading-edge concentration gradient and a long tail, the latter being caused as much or more by the reversed bottom current and vertical circulation of the tracer into the surface layer at the dead-end of the canal, as by its being trapped and slowly released aong the sides of the channel. This

type of dye distribution was frequently observed by the Hydraulic Laboratory in the results from measurements taken for the calculation of the longitudinal dispersion coefficient.

7.2.1 Tidal Velocities

When wind and salinity gradient effects are observed to be negligible in a tidal canal, the measured vertical velocity profile is often logarithmic. Based on the volume of the tidal prism, V, upstream of the cross-section of interest, this profile is given [from Nikuradse, 1933; Walton, in Morris, Walton and Christensen, 1978, p. 227] by

$$u(z) = \frac{1}{A}\frac{dV}{dt}\frac{\ell n\ (29.73z/k)}{\ell n\ (29.73d/ke)} \tag{7.15}$$

where

V = upstream volume of tidal prism, (L^3)

e = exponential constant = 2.718

In the bottom layer of a vertically-4 layered model

$$u_b = \frac{1}{A}\frac{dV}{dt}\frac{\ell n\ (29.73\ d_b/ke)}{\ell n\ (29.73\ d/ke)} \tag{7.16}$$

where

d_b = thickness of bottom layer, (L)

and the mean velocity in any other layer, V, from depth $z=d_1$ to $z=d_2$, is given by

$$u = \frac{1}{A}\frac{dV}{dt}\frac{(d_2\ \ell n\ (29.73\ d_2/ke) - d_1\ \ell n\ (29.73\ d_1/ke))}{(d_2 - d_1)\ \ell n\ (29.73\ d/ke)} \tag{7.17}$$

Using the above formulae, the depth-dependance of the expression leads to a lateral variation of velocity in trapezoidal channels. The variation of depth with tide is implemented by expanding and contracting the sizes of the cells in the computational network according to the

change in cross-sectional area. The change in the vertical dimension is the same for all cells in a given layer, but in the lateral direction the change in cell width is made proportional to its mean velocity vector, and the sum of all the expansions or contractions of the cells in a given cross-section during one time-step is equal to the change in the cross-sectional area of the flow.

7.2.2 Wind-Induced Circulation

Traditionally, the effect of wind over a waterbody has been modeled using closed-form functions or empirical formulae. These are derived from the turbulent momentum equation in the longitudinal x-direction in a semi-enclosed basin, assuming constant density

$$\frac{du}{dt} = g\frac{\partial\eta}{\partial x} + fv\frac{\partial}{\partial x}\frac{\tau_{xx}}{\rho} + \frac{\partial}{\partial y}\frac{\tau_{xy}}{\rho} + \frac{\partial}{\partial z}\frac{\tau_{xz}}{\rho} - \frac{1}{\rho}\frac{\partial P_a}{\partial x} \tag{7.18}$$

where

η = elevation of water surface, (L)

v = lateral velocity, (L/T)

f = Coriolis parameter, (1/T)

τ_{xx}, τ_{xy}, τ_{xz} = shear stress in x,y, and z directions respectively, with respect to the x-direction, (M/LT^2)

ρ = density of fluid, (M/L^3)

P_a = atmospheric pressure, (M/LT^2)

Assuming quasi-steady flow, constant atmospheric pressure, small longitudinal and lateral shear stress gradients, and neglecting the Coriolis effect, Equation (7.18) becomes

$$g\frac{\partial\eta}{\partial x} = \frac{\partial}{\partial z}\left(\frac{\tau_{xz}}{\rho}\right) \tag{7.19}$$

Then, for a rectangular, prismatic dead-end canal with constant water slope

$$\frac{\tau_{xz}}{\rho} = gz \frac{\partial \eta}{\partial x} + a_o \tag{7.20}$$

where

a_o = a constant of integration

For the case of a semi-enclosed waterbody in which the lateral shear stress is negligible compared to the longitudinal shear stress, the classical form of the latter is

$$\tau_{xs} = K_w \rho w_s^2 \cos\theta |\cos\theta| \tag{7.21}$$

where

K_w = drag coefficient, (dimensionless)

θ = angle between wind and positive longitudinal direction of canal reach, (degrees)

w_s = wind speed, (L/T)

τ_{xs} = surface longitudinal wind shear stress, (M/LT^2)

Wu's [1969] form of the wind drag coefficient, K_w, is used in this model

$$\begin{aligned} K_w &= 0.6 \times 10^{-6}\, w_s^{1/2} && \text{for } w_s < 49.2 \text{ fps} \\ &= 3.1 \times 10^{-6} && \text{for } w_s > 49.2 \text{ fps} \end{aligned} \tag{7.22}$$

The vertical eddy viscosity coefficient, or vertical momentum transfer coefficient, N_z, can be related to the vertical component of the longitudinal turbulent shear stress, τ_{xz}, using the Fick's first law analogy, as

$$\tau_{xz} = \rho N_z \frac{\partial u}{\partial z} \tag{7.23}$$

In many numerical models the vertical momentum coefficient has been assumed to be constant. Using this assumption, Heaps [1972, 1974] and Wang and Connor [1975] achieved fairly consistent results. However, since the turbulent velocity fluctuations, w', at the bed may be assumed to be zero, and since $\partial u/\partial z$ is quite large near the bed, N_z must be very small at that location. At the surface, while some investigators assume that N_z is zero, or that w' is zero, there is usually some vertical surface activity in almost all open waterbodies. It may thus be assumed that N_z will also be small, but not usually zero, at the surface, and these observations lead to the classical parabolic form of the coefficient

$$N_z = 0.4\ u^* \frac{z}{d}(1-\frac{z}{d}) \tag{7.24}$$

which has been substantiated by Jobson and Sayre [1970] and Pritchard [1950] in both laboratory and field experiments.

However, when the parabolic form is incorporated into the analytic solution for the wind-induced velocity field, two new coefficients are introduced. Both of these coefficients would have to be evaluated with field data if such a form were adopted. It was observed, however, from the form of the velocity profile resulting from the solution of equation (7.23) for $\partial u/\partial z$, its integration to obtain u(z), and the application of the condition expressing zero net flow through any cross-section

$$\int_o^d u(z)dz = 0 \tag{7.25}$$

that the vertical momentum transfer coefficient could reasonably be expressed as a function of the total depth d

$$N_z = \bar{N}_z\ d \tag{7.26}$$

where

$\bar{N}_z$ = a canal network constant defined by the above equation, Equation (7.26), (L/T)

The equation for the wind-induced vertical velocity profile developed in conjunction with the vertical momentum-transfer coefficient is

$$u(z) = \frac{K_w w_s^2 \cos\theta|\cos\theta| z(\frac{3z}{d} - 2)}{4N_z} \tag{7.27}$$

This equation is the same as that given by Cooper and Pearce [1977] with a change of axis, and has the parabolic form shown in Figure 7.2.

Equation (7.27) can be easily modified for a trapezoidal, prismatic dead-end canal, by multiplying the function by a ratio of top width, B, to the width of the channel at the desired elevation

$$u(z) = \frac{\bar{B}(z) K_w w_s^2 \cos\theta|\cos\theta| z\ (\frac{3z}{d} - 2)}{4N_z} \tag{7.28}$$

where

$$\bar{B}(z) = \frac{b + d(s_L + s_R)}{b + z(s_L + s_R)} \tag{7.29}$$

b = bottom width, (L)

s_L, s_R = inverse side slope of left and right banks respectively, (dimensionless)

For any layer, k, the wind-induced vertical velocity profile may then be written

$$u_k = \frac{\bar{A}(k)\ K_w w_s^2 \cos\theta|\cos\theta|\ [d_2^2\ (\frac{d_2}{d} - 1) - d_1^2(\frac{d_1}{d} - 1)]}{4\bar{N}_z\ d(d_2 - d_1)} \tag{7.30}$$

where

$\bar{A}(k)$ = cross-sectional area correction factor for layer k

$$= \frac{[2b + (d + d_{-1})\ (s_L + s_R)]}{[2b + (d_1 + d_2)\ (s_L + s_R)]} \frac{d - d_{-1}}{d_2 - d_1} \tag{7.31}$$

where

d_1 = elevation of base of layer k, (L)

d_2 = elevation of top surface of layer k, (L)

d_{-1} = elevation of base of top layer, (L)

A typical measured velocity from the 57 Acres canal network is shown in Figure 7.3. Superimposed on the observed data are two plots representing the vertical velocity distribution without (Equation 7.27) and with (Equation 7.28) the width correction factor. The correction factor gives a good fit between the theoretical and observed profiles, and fitting the profiles is one method for calibrating the N_z and $\bar{N}_z$ coefficients used in the three dimensional model.

It is known that there is some time lag involved in setting up the wind-induced circulation after a change in the wind. No useful discussion of this aspect could be found in the literature, and unfortunately the period over which wind data were recorded in the 57 Acres canal System was not long enough to identify the differences between fully-developed and partially-developed conditions. The theory described above is for a fully-developed flow and therefore the model can only be theoretically calibrated if the measured flow is also fully developed. A further complication is the observation that any fully-developed set-up would have to be completed within one quarter tidal cycle, because the tide reversal would tend to break-up whatever set-up had been achieved. To approximate the process of setting up a wind-induced flow, a time constraint can be specified over which the wind speeds and

directions are averaged and a resultant longitudinal wind vector is obtained. This parameter could also be used in the calibration of the model.

7.2.3 Secondary Currents

Secondary currents in straight channels were first observed by Stearns in 1883 (according to Ikeda and Kikkawa [1976, p. 1]). Nikuradse [1930, in Ikeda and Kikkawa, 1976, p. 1] measured the velocity in noncircular ducts and found comparatively large velocities in the corners. Prandtl intuitively explained that these higher currents may result from the transport of momentum by the secondary flows from the center of the flow toward the boundaries. Ikeda and Kikkawa [1976, p. 12] explained that the secondary flow is produced by the non-uniform distribution of turbulent velocity fluctuation differences $(\overline{v'^2} - \overline{w'^2})$ in the cross-sections. They found that, in a wide channel,

$$\frac{v}{u^*}, \frac{w}{u^*} > 0.2 \tag{7.32}$$

where

v = lateral velocity component, (L/T)

w = vertical velocity component, (L/T)

which, for Floridian canals can be shown to give values on the order of 0.03 fps for tidal velocites on the order of 0.1 fps. Since for sloping banks these values will be even smaller, secondary currents in straight reaches were not considered in this model.

Rozovskii [1957] studied the flow of water in bends, considering a logarithmic vertical velocity profile and deriving the radial velocity component and an expression for the distance over which this component will decay after passing the end of the curve. Since his work did not

specifically treat velocity fields with flow reversals due to wind set-up or density-induced flows, it was necessary to find whether this work could be extended to this type of flow. However, no such work could be found, and since the data from Hydraulic Laboratory field work were limited in this regard, an attempt was undertaken to explain this phenomenon from basic physical principles.

In the analysis of wind-induced circulation it was assumed that a condition of zero net flow would exist at any cross-section in non-tidal flow, which is argued from the horizontal water surface assumption. Extending this concept to secondary flows, it can be assumed that the rotational flow in the top layers will tend to cancel that in the lower layer, as the rotational direction of each layer and the momentum associated with each layer is the same. It was also assumed that extending the validity of the superposition principle for these flows would be valid, so that the only nonzero net effect would be that which is due to tidal flow, which is assumed to be logarithmic in the vertical and therefore consistent with Kozovskii's analysis.

Rozovskii [1957, Ch. 2], considering the linear shear stress distribution that results is a logarithmic vertical velocity profile in turbulent flow in a wide channel, developed the following expression for the radial component of velocity

$$v = \frac{ud}{\chi^2 r} \left[F_1(\eta) - \frac{\sqrt{g}}{\chi C} F_4(\eta)\right] \tag{7.33}$$

where

χ = von Karman's constant

= 0.4, (dimensionless)

r = radius of bend, (L)

η = dimensionless depth

= z/d

C = Chezy's coefficient, $(L^{1/2}/T)$

$$F_1(\eta) = 2\int \frac{\ln \eta}{\eta - 1}\, d\eta \tag{7.34}$$

$$F_4(\eta) = \int \frac{\ln \eta}{\eta - 1}\, d\eta + 0.8\,(1 + \ln \eta) \tag{7.35}$$

Using Chezy's equation, Manning's equation, and the assumption that $(R/k)^{1/6}$ is approximately equal to 1, which is valid for Floridian canals, Chezy's coefficient may be written

$$C = 8.25\,\sqrt{g} \tag{7.36}$$

and Rozovskii's expression for the radial velocity component becomes

$$v = 6.25\, u\frac{d}{r}\,[F_1(\eta) - F_4(\eta)/3.3] \tag{7.37}$$

Equations (7.34) and (7.35) may be solved by numerical integration, which for a layered model results in

$$\bar{F}_1(k) = \frac{\left[2\sum_{i=2}^{\infty} \frac{(-p)^i}{i^2\,(i+1)} + 1.29p\right]_{\eta_1}^{\eta_2}}{\eta_2 - \eta_1} \tag{7.38}$$

where

$\bar{F}_1(k)$ = average value of $F_1(\eta)$ integrated over layer k

η_1, η_2 = lower and upper dimensionless depth of layer

$p = \eta - 1,\ 0 < p \leq 1$

The function $F_4(\eta)$ was fitted with a parabola, the form of which was obtained from the given values of $F_4(\eta)$ at η of 0.1, 0.5, and 1.0, so that when layer averaging was performed, errors would tend to cancel. The layer-average form of Equation (7.35) is thus given by

$$\bar{F}_4(k) = \frac{\left[-2.88\eta + 4.68\eta^2 - 1.73\eta^3\right]_{\eta_1}^{\eta_2}}{\eta_2 - \eta_1} \tag{7.39}$$

The form of the radial velocity given by Equation (7.37) applies only in the length of the curved portion of the bend. Downstream this velocity decays almost to zero over a distance, L_d, given by Rozovskii [1975, p. 111] as

$$L_d = \frac{C^2 d}{2g} \ln \left(\frac{X}{p}\right) \tag{7.40}$$

where

p = permissable deviation from the background velocity.

For this model p was chosen to be 0.2. Writing Chezy's C in terms of Nikuradse's k, Equation (7.40) becomes

$$L_d = \frac{8.25^2 d}{2} \ln (0.4/0.2) \tag{7.41}$$

$$= 102 \text{ d}$$

With this form, the radial velocity at any point located a distance x away from the curved portion of the bend, and less than the decay distance L_d, is

$$v(x) = v \exp (-3x/L_d) \tag{7.42}$$

This velocity is averaged over the length of any cell in which it acts, giving the layered-model form of the lateral velocity due to secondary currents.

A comparison between theoretical and observed radial velocities (Figure 7.4) shows good agreement. In this case, the observed radial velocities were measured just downstream of the crown in the South Loop of the 57 Acres canal network (Figure 6.33) where the three velocity towers were located. The theoretical velocities were obtained from

Equation (7.37). In each case, velocities on the order of 0.05 fps were measured, which is sufficient to cause a considerable overturn of the flow.

7.2.4 Density-Induced Currents

In some canals in Florida there exist density gradients which could significantly affect the flow regime. In some cases, as in the Loxahatchee River canals, stratification can develop and a density wedge can be observed intruding and retreating with the tide. The forcing function for a wedge is the rising saltwater surface in the connecting waterbody. As the tide recedes, the saltwater surface in the receiving waterbody falls and the wedge receeds as it loses the potential energy gained during the flood tide. In the presence of a wind the mechanism of the salt wedge becomes more complex. In a dead-end canal the entering wedge may impinge on the dead-end, thus altering the characteristics of the flow. Also, if a wedge enters on the food tide it may encounter the remnant of a salt dome remaining from a previous tidal cycle. In the latter case, the depth d becomes difficult to define as the velocity field associated with the wedge acts throughout the height of the salt water.

Most numerical models of density gradient or stratified flow model only the continuity of salt through a mass-balance equation, neglecting the effect of density gradients on the continuity and momentum equation [King, Norton, and Orlob, 1973; Hess and White, 1974]. This appears to be a reasonable approach for continuously stratified flow, but cannot be applied to a salt wedge in a canal because of the discontinuity at the interface between the fresh and salt water. Most of the work to date on analysis of the movement of a salt wedge is based on empirical formulae

[Keulegan, 1958], or substantially simplified equations for the movement of the front of the wedge. Models which do incorporate the density terms, such as that by Leendertse and Liu [1975] are very expensive to run, and their accuracy in a tidal canal would be doubtful because of the nature of the unknowns, such as bed and bank roughness. An empirical approach was therefore used to describe the salt wedge phenomenon.

The method finally employed is based on the assumption that the mean velocity at the tidal entrance, $u_{sm_{TE}}$, is proportional to the rate of rise of the interface at the entrance

$$U_{s_{m_{TE}}} = u_4 \frac{dd_s}{dt} \tag{7.43}$$

where

u_4 = an empirical coefficient, (dimensionless)

d_s = depth of the salt water wedge, (L)

Assuming the shape of the salt water wedge to be triangular, (Figure 7.5), which is the simplest shape, a set of equations may be developed for the movement of the wedge through the reaches of a canal network, both before and after it impinges on a dead-end. The length of the wedge, L_w, before it arrives at a dead-end is given by

$$L_w = 2\ u_4 d_{s_{TE}} \tag{7.44}$$

where

$d_{s_{TE}}$ = elevation of saltwater interface at tidal entrance above its low tide value, (L)

and the depth at any point, d_s, is

$$d_s = d_{s_{TE}} \left(1 - \frac{x'}{2\ u_4\ d_{s_{TE}}}\right) \tag{7.45}$$

where

x' = distance from tidal entrance, (L)

The mean velocity in the wedge, u_{sm}, at x' is

$$u_{sm} = u_4 \frac{dd_s}{dt} \tag{7.46}$$

The vertical velocity profile in the wedge is given by

$$u_s(z) = 3u_4 \frac{dd_s}{dt} \left(\frac{z}{d_s}\right)^2 \tag{7.47}$$

and the velocity in the freshwater layer is given by

$$u_f = \frac{-u_4 \frac{dd_s}{dt}}{\frac{d}{d_s} - 1} \tag{7.48}$$

Similar equations for the wedge after it has impinged on the dead-end of the canal, and for the movement of the wedge on the ebb tide, are given by Walton in Morris, Walton, and Christensen [1978, pp. 252-254].

Figure 7.6 shows that reasonably good agreement was obtained between measured values and model results for the Loxahatchee River north canal. It will be shown in section 7.5.3 that the transient measured dye concentration profiles are also fairly well followed. Since comprehensive data were not available for other locations, the calibration value of the coefficient u_4 could not be checked. Since this coefficient may be dependent on the length of the canal, further study will be required to determine its variability.

7.2.5 Three-Dimensional Mass-Transport Coefficients

The three-dimensional mass-transport equation for turbulent flow in a homogeneous waterbody, Equation (7.4), has been derived in a number of references and will be used as the starting point for the following

discussion. This section is devoted to the derivation of the model forms of the diffusion coefficents in the three coordinate directions.

7.2.5.1 Diffusion and Dispersion Coefficients and Transport Mechanisms

A *diffusion* coefficient has been defined as a coefficient representing transport, or more precisely a transport analogy, which is averaged over a time scale associated with the transport mechanism, such as a molecular time scale or a turbulent time scale. A *dispersion coefficient* is defined as a coefficient representing a transport analogy which is averaged over a length scale associated with the transport mechanism. Thus, in a one-dimensional model a longitudinal dispersion coefficient was obtained by averaging the three-dimensional mass transport equation over a cross-sectional area. Similarly, dispersion coefficients can be obtained by averaging over one coordinate direction, or over the length scale of a computational cell. This diffusion may be represented by dispersion coefficients in a cellular model, and their associated dimensionless coefficients obtained by calibration with field data.

The spreading of a substance in a flow field involves two separate mechanisms, convection (or advection) and diffusion. By definition, *convection* is the transport of a substance with the velocity of the fluid, while *advection* is more generally the transport in the direction of the flow. In this work the more precise term, convection, is used.

The turbulent diffusion coefficients E_x, E_y, and E_z were obtained in the derivation of the turbulent form of the three-dimensional mass-transport equation, when Fick's first law was applied to relate turbulent fluctuation cross-products, such as u'c', to concentration gradients. Thus, it was assumed that an analogy of the form

$$\overline{u'c'} = E_x \frac{\partial c}{\partial x} \tag{7.49}$$

is valid, and the turbulent diffusion terms on the right hand side of expressions similar to Equation (7.49) were substituted for the cross-product terms.

This concept can be extended to the discretization of a numerical model. Analogously to the mechanism of turbulent diffusion, spatial variations in the velocity field with length scales smaller than the grid size can become lost in the definition of a mean velocity within the grid, unless another dispersion term is introduced to account for this effect by averaging over the computational cell. Another use of the diffusion term is to account for unknowns in the velocity field, such as the circulation remaining at slack tide. Holley and Harleman [1965], for example, were among the first to investigate the mechanism of dispersion in oscillatory flows, and developed a form of the longitudinal dispersion coefficient based on Taylor's [1954] and Elder's [1959] work

$$E_\ell = 20.2\ Ru^*(t) \tag{7.50}$$

Theoretically, assuming no external forcing functions except the tide, E_ℓ becomes zero at slack tide. However, it has been observed that in tidal canals, particularly near dead-ends, there is still considerable water movement, and therefore transport, due to eddies of various sizes and residual momentum effects. With this in mind, a more realistic form of Holley and Harleman's oscillatory dispersion coefficient may be

$$E_\ell = 20.2\ Ru^*(t) + E_o \tag{7.51}$$

where

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

It has been found to be convenient to include such a coefficient in the model, as will be described in Section 7.2.5.3.

7.2.5.2 Longitudinal and Lateral Diffusion Coefficients

The choice of the model forms of the diffusion coefficients is complicated by the existence of many empirical formulations. Since the bottom and side effects of a canal are recognized as an important part of the diffusion mechanism, the work of Taylor [1953, 1954], Elder [1959], and Aris [1956] will be followed in subsequent derivations since their work was more directly affected by the boundary roughness.

Taylor [1954] showed that for a flow which obeyed the "universal" distribution of velocity in a pipe, the longitudinal diffusion coefficient, E_x, could be written

$$E_x = 0.104\ Ru^* \tag{7.52}$$

This form should theoretically hold near any solid boundary, and in addition is assumed to hold in any velocity field with logarithmic profiles. Since in tidal canals the non-logarithmic profiles due to wind shear and density gradients are found principally in the vertical direction, it will be assumed that Equation (7.52) is valid for any horizontal plane in the flow.

For flows induced by density gradients the form of the interface is a function of both x and t. However, the interface is a discrete plane at which one homogeneous fluid of density ρ_o is assumed to change to a second homogeneous fluid of density $\rho_o + \Delta\rho_o$. If it is assumed that equation (7.52) holds for each fluid separately, then the solutions can be matched across the interface. Thus, in all cases the longitudinal diffusion coefficient, E_x, can be written,

$$E_x = \frac{K_x du}{8.25} \tag{7.53}$$

The lateral diffusion coefficent has been traditionally related to the longitudinal form by a constant of proportionality. For example, Elder [1959] suggested the expression

$$E_y = 0.23\ Ru^* \tag{7.54}$$

which is a dispersion coefficient because it is depth-averaged. The value 0.23 was obtained from laboratory experiments, and was confirmed from field results by Fischer [1967b, p. 191] and others. For flow in bends, Fischer [1969, p. 500] found substantially larger values for E_y/Ru^*, up to 2.4, or ten times the value for straight channels.

The lateral transport mechanism is analogous to that for longitudinal transport, that is, transverse deviations from the mean velocity produce transverse dispersion in the same way that longitudinal deviations from the mean velocity produce longitudinal dispersion. Assuming a logarithmic velocity profile based on the mean depth in the section of flow being considered, the mean velocity, v, from one cell to a laterally adjacent cell will be governed by the velocity difference between cells

$$v \propto (u_j - u_{j-1}) \tag{7.55}$$

where

u_j, u_{j-1} = velocities in laterally adjacent cells, (L/T)

Assuming logarithmic velocity profiles

$$v \propto u^* \ln \left(\frac{d_{m_i}}{d_{m_{i-1}}}\right) \tag{7.56}$$

and therefore

$$v \propto u^* \tag{7.57}$$

It may be assumed that lateral velocity profiles are logarithmic from the banks to the centerline, even though vertical velocity profiles may not be logarithmic. Accordingly, the following expression has been used for the lateral diffusion coefficient

$$E_y = K_y du/8.25 \tag{7.58}$$

It has been suggested (Section 7.2.5.1) that a background diffusion coefficient could be used to account for the effects of unknown residual eddies and currents around the times of slack tides. However, the field data are not complete enough to permit a rigorous development of this concept in three coordinate directions. Accordingly, it was decided to assume that this coefficient is isotropic and to find a value by means of model verifications. The model forms of the longitudinal and lateral diffusion coefficients may be written

$$E_x = K_x du/8.25 + E_o \tag{7.59}$$

$$E_y = K_y du/8.25 + E_o \tag{7.60}$$

7.2.5.3 Vertical Diffusion Coefficient

The theory used above to develop the longitudinal and lateral diffusion coefficients cannot be used for the vertical diffusion coefficient since that theory is based on logarithmic velocity profiles. It was decided, therefore, to develop an expression from Prandtl's mixing length theory, from which a vertical diffusion coefficient of the form

$$E_z = K_z \ell^2 \left|\frac{du}{dz}\right| + E_0 \tag{7.61}$$

can be written, where

K_z = vertical dimensionless diffusion coefficient

This coefficient may be further developed into a form suitable for the numerical model by integrating over the thickness, d_ℓ, of the layer k, from $z = d_1$, to $z = d_2$. The integral form, after integration, becomes

$$E_k = \frac{K_z \ell^2}{d_\ell} \left| u(d_2) - u(d_1) \right| + E_o \tag{7.62}$$

In most studies of diffusion or dispersion the value of the mixing length, ℓ, is based on some uniform, geometric, characteristic length scale such as the mean tidal depth or the width of the reach. Since the parabolic wind profile, Equation (7.28), has a maximum at two-thirds the depth and a turning point at one-third the depth, these depths are the logical choices for length ℓ. From Equation (7.15), which is the expression for the vertical tidal velocity profile in terms of the upstream tidal prism, Equation (7.28) giving the vertical velocity profile in wind-induced flow with the top-width correction factor, and Equation (7.31) for the cross-sectional area correction factor, the model form of the vertical dispersion coefficient for each layer is given by

$$E_k = \frac{K_z d^2}{9 d_\ell} \left| \frac{1}{A} \frac{dV}{dt} \frac{\ln(d_2/d_1)}{\ln(29.73\ d/ke)} \right.$$

$$\left. + \frac{0.25\ K_w w_s^{\ 2} \cos\theta \left|\cos\theta\right| (d_2 - d_1)\left[\frac{3}{d}(d_1 + d_2) - 2 \right.}{N_z} \right| \tag{7.63}$$

$$+ E_o$$

For the case of tidal flow with wind shear, and a density current induced by a saline wedge, an expression similar to Equation (7.63) can be derived by adding some of the expressions in Equations (7.47) and

(7.48), and expressions in Equations (4.71) through (4.83) in Morris, Walton, and Christensen, [1978, pp. 252-254], to Equation (7.63). This form of the equation satisfies Equation (7.61) except at the interface between the two fluids. At the interface an extra term is required to account for the decrease in interfacial mixing, as the density difference between the two fluids increases. The rate of mixing across the interface is usually considered to be governed by the Richardson number, Ri, defined as

$$Ri = \frac{g}{\rho} \frac{\partial\rho/\partial z}{(\partial u/\partial z)^2} \tag{7.64}$$

The vertical diffusion coefficient averaged over the layer containing the interface, $\tilde{E}_k$, is then written

$$\tilde{E}_k = E_k(1 - 0.1\ Ri)^{1/2} \tag{7.65}$$

where the term $(1 - 0.1\ Ri)^{1/2}$ is obtained from Obukhov [1971] through a comparison by Blumbeg [1977]. This form was found to give satisfactory results for the three-dimensional model.

7.3 Development of Three-Dimensional Numerical Model

One of the principal goals of the development of the numerical model was to produce a computer program which would be accurate, reasonably economical, and relatively simple to use. This required that the number of input parameters, exclusive of geometry and forcing-functions or boundary conditions, should be minimized. As can be seen from Sections 7.1 and 7.2, the only coefficients required for calibrating the model are the dimensionless dispersion coefficients, the vertical momentum transfer coefficient, and the dimensionless velocity associated

with the saline wedge. The solution technique is the method of second moments, described in Section 7.1.5.5.

The program, called CANNET3D (from Canal Network, 3-Dimensional), was written in Fortran IV for an Amdahl 470 computer, which is an outgrowth of, and similar to, the IBM 370. The program requires just over 200K bytes of storage and has a number of subroutines, the names of which will be designated in capital letters in the remainder of this text. Program variables will be designated in the same manner.

A user's manual, flow chart, and program listing for the program are contained in Appendices E and F in Morris, Walton, and Christensen [1978].

In this computer program several of the variables have subscripts which are a combination of the letters i, j, and k, representing spatial reference points in the x (longitudinal), y (lateral), and z (vertical) coordinate directions, respectively. Thus i will denote the number of a segment in a reach, j will denote the lateral location from the left bank, and k will denote the number of the layer beginning at the bed. If a variable has all three subscripts it is a function of the three independent coordinates, whereas if it has only one or two subscripts its value is uniform with respect to the other coordinate direction(s). Thus, the longitudinal cell length is written Δx_i, in segment i, as its length is independent of its location in the cross-section. Similarly, the thickness of a layer is written Δz_k, since it is independent of width and length.

The two superscripts, n and n+1, are used to denote the time level as a multiple of the time interval, Δt. The time level n is defined as the last time level at which all the variables were calculated. Thus,

time level n+1 is the time level currently being evaluated. If these superscripts are omitted, the expressions can be applied at any time level $n\Delta t$.

A wavy bar, ~, is written above variables which are calculated during the time level n+1 as an intermediate step to the final value of that variable. The subscript, av, is used where values are averaged between time levels n and n+1.

7.3.1 Layout of Geometry

The first step in numerically evaluating an existing or proposed canal network is to arrange the geometry in the form required for input into the model. The geometry is expressed in terms of reaches, junctions, dead-ends, tidal entrances, and a special kind of junction called a null point (Figure 7.7).

Each reach is assigned a consecutive number from 1 to NREACH (the toal number of reaches in the system, including reaches formed by the introduction of a null point), in any order desired. This procedure reduces the number of changes necessary to be made in the program input specifications with design changes. The reaches are rearranged in the subroutine ORDER, retaining their original identification number, in a sequence beginning with the null-points and dead-ends of the canal network and proceeding to the tidal entrances. In this rearrangement process, at any junction the upstream reaches of the junction are ordered before the downstream reaches. This rearrangment is necessary for efficiently calculating the incremental tidal prism volume from the upstream limits to the tidal entrance, and the new distribution is stored in the array NRCH(*). (NOTE: the number of asterisks in paren-

thesis after the name of an array indicates the dimensional order of the array).

A junction of the model canal network may be either a null point, an area of the network at which one or two branch canals join a main canal, a theoretical zone at which two portions of a canal reach with differing geometric characteristics join, a dead-end, or an enlarged water area which has been called a "lake" in this model. Once the locations of the junctions have been determined, a reach between two hydraulically adjacent junctions is defined to be a trapezoidal, prismatic channel.

Each dead-end junction, except a lake, is assigned the number 1. Each interior junction (those excluding dead-ends, null points, lakes, and tidal entrances) are numbered from 2 to (NJUNC + 1), where NJUNC is the number of internal junctions in the network.

A lake is an enclosed basin with one entrance connected to a reach, which is substituted for a dead-end at the upstream limit of a branching network. The flow and concentration are *not* modeled in a lake, as this would require a separate, complex model. However, the volume of a lake is used in the calculation of the upstream tidal prism, which is then used to calculate the mean tidal velocity in downstream reaches. Thus, assuming the water level in the lake is the same as the water level throughout the canal network, a lake can be treated as a special case of the dead-end, with finite surface area, but having no effect on mass transport. The lakes are numbered from (NJUNC + 2) to (NJUNC + NLAKE + 1), where NLAKE is the number of lakes at the dead-ends of the system.

Tidal entrances are similar to junctions in that they can be joined by a main canal and one or two branches. They are numbered from

(NJUNC + NLAKE + 2), to (NUNC + NLAKE + NTES + 1), where NTES is the number of tidal entrances.

A null point must be defined if two tidal entrances are specified. Consider a system with two tidal entrances, in which the velocity field is produced entirely by changes of the tidal elevation in the connecting waterbody. Assuming there is no phase difference between the entrances, the flood flow from each entrance will meet at the null point, at which the tidal velocities will be zero. If the flow is permitted to recombine and proceed up a branch canal, then a null point cannot be specified at that location. The formulation for locating a null point in an arbitrary network is beyond the scope of this work. However, in the case of a symmetric network a null point can be specified, provided there are no flow-dividing or recombining points at which the flow cannot be determined by consideration of the upstream tidal prism. Once the null point has been located it is considered to be the meeting point of two dead end canals at which only wind and density-induced flows have an influence across the boundary. The two adjacent reaches are separately numbered and each is assigned an upstream junction at the null point. Each of these upstream junctions is assigned the negative value of the number of the opposite reach. For example, if reaches number 8 and 9 are joined at a null point, the upstream junction of reach number 8 is assigned junction number -9, and the upstream junction of reach number 9 is junction number -8.

Once the network layout has been established, the length, bottom width, mean tidal depth, left and right inverse side slopes, the numbers of the upstream and downstream junctions, Nikuradse's equivalent sand roughness, and the alignment angle of each reach must be specified. The

alignment angle is referenced to true North at the site to permit the alignment of the longitudinal wind component. Similarly the length, width, mean tidal depth, and the identification numbers of the adjoining reaches are specified for each junction.

After the network geometry has been established, each reach should be divided into computational cells. The number of longitudinal divisions, NDIV(*), is specified with the input values for each reach, and the number of lateral, NLAYY, and vertical NLAYZ, divisions are specified as constants for the whole system. The three-dimensional model becomes two-dimensional when NLAYY is specified as 1, and three layers in the vertical are satisfactory for most simulations. The arrangment and numbering of computational cells is shown in Figure 7.8.

The junctions between reaches are used simply as transition zones from one reach to another, in which first and second moments of the pollutant mass are balanced but through which no diffusion is modeled. To match the lateral and vertical layering of reaches entering the junction perpendicular to each other, the junction is also divided into NLAYZ vertical layers, each layer having $NLAYY^2$ cells (Figure 7.9).

7.3.2 The Velocity Field

The velocity field is obtained by superposition of the individual tidal, wind-induced, salinity-induced, bend, and lateral inflow effects. Each has a slightly different treatment which is discussed in the following sections and in Section 7.3.4.

7.3.2.1 Tidal Velocities

The tidal velocities are generated by the rate of change of the surface elevation at the tidal entrance(s). The tidal elevation can be input in one of two ways. If the elevations can be considered to follow

a simple harmonic function, starting either at low or at high tide, the tidal amplitude, AMP, and the period, T, are read in and used in the expression

$$d = d_{o_{TE}} + a \cos \omega t \tag{7.66}$$

where

$d_{o_{TE}}$ = mean depth at tidal entrance, (L)

The tidal amplitude is made positive or negative depending on whether the simulation is to begin at high or at low tide. The second method for entering tidal values is to digitize the elevations with respect to the mean tidal elevation at each time interval. It is not necessary to create a new set of tidal input values if the time step, Δt, is changed, because a user-provided interpolation parameter, INTERP, calculates a linear interpolation between specified values.

The magnitude of the mean tidal velocity at any section is obtained from the upstream tidal prism volume. The calculations begin at the dead-ends of the network and proceed toward the tidal entrance(s). The vertical logarithmic velocity profile for any computational cell is based on the mean depth of the trapezoidal channel over the width of the cell. The mean depth is calculated in the subroutine MEAND, and the logarithmic velocity is then integrated over the vertical extent of the cell to determine the mean longitudinal velocity in that cell.

The cell structure of the model is flexible both laterally and vertically, such that the mass of fluid transported longitudinally between adjacent cells remains entirely within the receiving cell. The rate of change of the cross-sectional area of the receiving cell is proportional to the velocity into that cell, and the vertical layers respond to the cumulative flow from all the lateral layers as if that

layer were one cell. Thus, the depths of each cell in any layer, except the bottom one, will always be uniform.

Additional details dealing with the flexible cell structure and the forms of the equations for the area of, and velocity in, each cell are given in Morris, Walton, and Christensen [1978, pp. 291-293].

7.3.2.2 Wind-Induced Circulation

Similarly to the methods of specifying the tide for the model, wind speed and direction can be entered in one of two ways. If a constant vector wind is blowing then the constant wind speed, WSC, in mph, and constant wind direction, WANGC, in degrees, are read in and converted to fps and radians, respectively. Otherwise, the wind speed and direction can be digitized and entered at the same time step, and at the same time, as the tidal data. Again, the same INTERP parameter can be specified if the values need to be linearly interpolated.

The velocity induced by the wind in layer k is constant throughout the entire length and width of the layer. Also, its net flux through a cross-section of the reach is zero. Therefore, this velocity does not affect the cross-sectional areas of computational cells. The center of mass of each cell is moved a distance, X, equal to the sum of the component distances of the logarithmic tidal velocity, XT, and the wind-induced velocity, XW(*), in that layer,

$$X = XT + XW(NLZ) \tag{7.67}$$

where

NLZ = vertical layer number

and as the wind-induced velocity is uniform in each layer, there is no vertical transfer of mass to conserve mass.

The junction, as stated before, acts as a transition zone between reaches. A simple conservation of mass routine matches the conditions in the adjacent cells of adjacent reaches, by transferring mass vertically in the junction to ensure that the mass entering each layer of the junction equals the mass exiting. The process is distributed throughout the junction depending on the volume flow from each reach boundary cell. A similar vertical mass transfer is also performed at the dead-ends.

Since the theoretical expression for the wind-induced vertical velocity profile has a reversal at the two-thirds depth and a turning value at the one-third depth, some multiple of three vertical layers will retain the essential features of the distribution without cancellation in the cell-averaging process. In fact, for most practical work three vertical layers, and no more than six, will provide sufficient information for the design engineer.

7.3.2.3 Secondary Currents

Secondary currents, as described in Section 7.2.3, have no effect on the longitudinal velocity distribution. They simply redistribute the fluid mass through the radial velocity pattern expressed in Equation (7.37). A subroutine, HELIX, calculates this effect in the model.

For each layer k, at the crown of the bend, a layer-averaged velocity, v_{ck}, developed from Equations (7.37), (7.38), and (7.39), is given by

$$v_{ck} = 6.25 \frac{d}{r} u_i \, [\bar{F}_1(k) + \bar{F}_4(k)/3.3] \tag{7.68}$$

where

u_i = mean velocity due to tidal component in segment i, (L/T)

This velocity is constant in the layer, throughout the length of the bend, but decays in the direction of flow to within 2 percent of zero

according to Equation (7.42). Since the y-axis is defined from the left bank to the right bank, perpendicular to the longitudinal axis, specification of a positive bend radius defines flow around a bend to the left in a downstream direction. Similarly, if the bend radius is negative the flow is around a right bank bend.

The input data for the model specifies the location of the crown of the bend, the bend radius, and the length of bend (Figure 7.10). When a bend is located during the simulation, a numerical pointer defines the location of the first longitudinal segment of the flow in which the secondary current develops. From this point, in the direction of flow, Equation (7.68) holds throughout the length of the bend, after which the radial velocity decays over a distance, PL, specified in the model in accordance with Equation (7.41). For each longitudinal segment, i, an averaging procedure based on the length of the bend is performed giving the following cell-averaged velocities, v_{ik}, in layer k,

1. For segment i totally contained in the bend:

$$v_{ik} = v_{ck} \tag{7.69}$$

2. For segment i of total length x having length Δx_1 within the length of the bend:

$$v_{ik} = \frac{v_{ck}}{\Delta x} [\Delta x_1 + \frac{L_d}{3} [1 - \exp[- \frac{3}{L_d} (\Delta x - \Delta x_1)]]] \tag{7.70}$$

3. For segment i totally outside the length of the bend, but at least partially within the decay distance, L_d:

$$v_{ik} = \frac{v_{ck} L_d}{3\Delta x} [\exp(- \frac{3x_1}{L_d}) - \exp(- \frac{3x_2}{L_d})] \tag{7.71}$$

where

x_1, x_2 = distances in the direction of flow from the end of the bend to the closest and farthest points of segment i, respectively, (L)

In each layer of segment i a volume of fluid, v_{ijk} given by

$$v_{ijk} = v_{ik} \Delta t \Delta x \Delta z \tag{7.72}$$

where

Δz = thickness of layer, (L)

j = number of lateral cell

is transferred between laterally adjacent cells of the layer to conserve fluid mass. Once this transfer has been completed in all the cells of a segment of the reach, a vertical mass balance between cells on the outside of the reach is performed.

7.3.2.4 Density Currents

The circulation induced by density gradients is calculated in a manner similar to the calculation of the wind-induced circulation, in the subroutine WEDGE, except that the longitudinal variation of the velocity in each layer is taken into account by using vertical transfer to conserve mass. The net mass flux through any cross-section is zero, and thus the flow makes no contribution to the change in cross-sectional areas of the computational cells.

A simulation of the movement of a salt wedge in a canal network must start at low tide, since the initial salt wedge configuration is specified in terms of the elevation of the interface in each reach this elevation is the elevation of the low tide interface in the receiving waterbody. The velocities given in Section 7.2.4 are layer-averaged, the cross-sectional areas of the cells are adjusted according to the

distribution of tidal velocities, and the center of mass of each cell is moved a distance, X, which is the sum of the component distances of the tidal velocity, the wind-induced velocity, and the density current. Mass in each vertical column and each cell is conserved.

7.3.3 Dispersive Terms in the Transport Equation

Since in a three-dimensional canal model, using three vertical and three lateral layers, eight of the nine cells in any cross-section are boundary cells, the usual finite-difference method for dispersive terms could not be used and a solution scheme had to be found through rearrangement of the governing equation. Considering a one-dimensional diffusion equation of the form

$$\frac{\partial c}{\partial t} = \frac{\partial}{\partial x}(E_x \frac{\partial c}{\partial x}) \tag{7.73}$$

a variable, u_{D_i}, defined by

$$u_{D_x} = - \frac{E_x}{c} \frac{\partial c}{\partial x} \tag{7.74}$$

can be introduced into Equation (7.73) giving

$$\frac{\partial c}{\partial t} + \frac{\partial}{\partial x}(u_{D_x} c) = 0 \tag{7.75}$$

where U_D is a longitudinal velocity term called the diffusion velocity [Spaulding, 1976]. Similarly, diffusion velocities in the y- and z-directions can be defined

$$u_{D_y} = - \frac{E_y}{c} \frac{\partial c}{\partial y} \tag{7.76}$$

$$u_{D_z} = - \frac{E_z}{c} \frac{\partial c}{\partial z} \tag{7.77}$$

such that when applied to the three-dimensional diffusion equation the three-dimensional convection equation is obtained

$$\frac{\partial c}{\partial t} + \bar{u}_D \cdot \nabla c = 0 \tag{7.78}$$

where

$$\bar{u}_D = (u_{D_x}, u_{D_y}, u_{D_z})$$

The diffusion velocities are used to transfer mass between adjacent cells, and they apply in the boundary cells because the condition of zero-flux through the sides of the channel is satisfied.

7.3.3.1 Longitudinal Dispersion Term

An upwind finite-difference approximation to the longitudinal diffusion velocity, $u_{D_i}^{n+1}$, in segment i, can be derived by considering the transport of mass from cell ijk to cell i+1jk. A longitudinal dispersion distance, XD, defined as

$$XD = u_{D_x} \Delta t \tag{7.79}$$

is added to the other distance terms that are due to tidal velocity, wind-induced velocity, and the density current (Section 7.3.2.4) to give the final displacement of the center of mass of cell ijk in the time interval Δt.

7.3.3.2 Lateral and Vertical Dispersion Terms

Since there are no specific lateral or vertical velocity terms in the model because of the flexible grid structure, except for the case in which secondary flows are present, the lateral and vertical dispersion terms are treated directly as dispersive velocities defined by Equations (7.76) and (7.77), respectively, which transfer fluid between adjacent cells and simply conserve mass. Thus, in the lateral and vertical directions a straightforward upwind-differencing technique is used. Transfer occurs in the direction of the negative concentration gradient.

To incorporate vertical dispersion in the presence of a saltwater wedge, the vertical dispersion coefficient, E_z, is multiplied by a function of the Richardson number, Ri, as defined in Equation (7.64). Dispersion is introduced across the salt wedge interface by assuming that the thickness of a vertical layer defines the distance over which the density varies from ρ_o to $\rho_o + \Delta\rho$. From Equation (7.64) the Richardson number can be written

$$Ri = \frac{-g}{\rho_{av}} \frac{\Delta\rho_o/\Delta z}{(du/dz)^2} \tag{7.80}$$

where

ρ_{av} = average density between the two layers, (M/L^3)

The value of du/dz can be calculated exactly from the superimposed expressions for the velocity field. The function $\phi(Ri)$ is evaluated from the expression

$$\phi(Ri) = (1 - 0.1\ Ri)^{1/2} \tag{7.81}$$

which is part of Equation (7.65), and used to modify the finite difference equation for the mass flux, cu_D, between vertical layers, provided the interface lies within one-half cell depth from the plane separating vertically adjacent cells. The mass flux may then be written

$$cu_{D_k} = \frac{1}{2\Delta z_k} (1 - 0.01\ R_i)^{1/2} (E_{z_{ijk+1}} + E_{z_{ijk}}) (c^n_{ijk+1} - c^n_{ijk}) \quad \text{for } Ri \leq 10$$

$$cu_{D_k} = 0 \quad \text{for } Ri > 10 \tag{7.82}$$

7.3.4 Lateral Inflows

The term representing the rate of production or loss, r_p, in the three-dimensional mass-transport Equation (7.4) was defined by Equation (7.6) for the one-dimensional model. This form can be extended to three-dimensions by defining the lateral inflow per unit length, $q_{I_{ijk}}$, for cell ijk. If the concentration of this inflow is $C_{I_{ijk}}$, then the rate of production or loss term is

$$r_{p_{ijk}} = \frac{q_{I_{ijk}} c_{I_{ijk}}}{A_{jk}} \tag{7.83}$$

The lateral inflow is added to the mass in cell ijk before the convection step. This is done so that the lateral inflow rate, q_I, can be added to the tidal prism volume to calculate the velocity through the cell during the current time step. Thus, the sum of the lateral inflows in segment i is added to the upstream tidal prism volume, and this value is then used to determine the cell-averaged tidal velocities and the change in the cross-sectional areas of cells in each segment. Thus the inflow conditions remain in phase with the solution scheme, i.e., the inflow concentrations are convected with induced velocities at the same time level, not the following time level.

The lateral inflow may be input either as a constant, or into any combination of cells on a time-varying basis. If the inflow rate and concentration remain constant throughout the simulation, then for each such inflow, OPT2(*) is set to 0, and the reach number, NR, the segment number NPX, the number of the cell in the cross-section, NC (based on the coordinate system shown in Figure 7.8), the lateral inflow rate QIC(*) and the concentration, CIC(*), are input and stored. If the

lateral inflow and/or its concentration are variable, OPT2(*) is set to 1 and only the variables NR, NDX, and NC are initially read to determine the location(s) of the inflow. During the simulation, the variable inflow data are read in at each time interval at which tide and/or wind values are input, and the INTERP routine will linearly interpolate the values as necessary.

7.3.5 Decay Coefficients

The three-dimensional mass-transport equation (7.4) describes the transport of a conservative, passive substance in a velocity field. Many substances, however, have some associated decay due to their chemical or biological structure or due to the effects of external phenomena. BOD, for example, has both a carbonaceous and a nitrogenous decay rate. The sinks of dissolved oxygen, besides the biological oxidation associated with BOD, include benthal decomposition of bottom deposits, the respiration of aquatic plants, and chemical oxygen demand, all of which may be characterized as decays. Water tracing dyes exhibit photochemical decay, and their absorption by banks or suspended solids acts as a variable decay. A conservative substance is considered to be one without any associated decays.

The many forms of decay can often be treated as first or higher order phenomena. The first-order exponential decay described by

$$\tilde{c}_o = c_{RW} + (c_o - c_{RW}) \exp(-Kt) \tag{7.84}$$

where

$\tilde{c}_o$ = concentration after a decay period t, (dimensionless)

c_{RW} = background concentration, (dimensionless)

c_o = initial concentration, (dimensionless)

K = decay coefficient, (1/T)

Differentiation with respect to time results in the expression

$$\frac{\partial c_o}{\partial t} = -Kc \tag{7.85}$$

As equations (7.4) and (7.85) are linear, they may be superimposed to produce the three-dimensional mass-transport equation for a non-conservative substance, which is equation (7.4) with the term -Kc added to the right hand side.

In the numerical model a distinction is made between two different forms of decay. The first is one which exists throughout the volume of a reach, while the second is one which only exists in certain portions of the reach. The reach uniform decay coefficient, K (in Equation 7.84), is input with the other reach data, while the local decay coefficients, which are assumed to remain constant throughout the period of the simulation, are input in a manner similar to the input of lateral inflows. For each coefficient its reach number, NR, segment number, NDX, cell number in the cross-section, NC, and decay coefficient, DECAY(*) are read into the program when the value of OPT for each set of data is nonnegative.

For economy of computing time, the decay over a time interval Δt is computed immediately after the convective step. A finite-difference form is used to approximate Equation (7.85).

7.3.6 Boundary Conditions

Since the three-dimensional mass-transport equation (7.4) is a second-order parabolic equation, a solution requires one set of initial conditions at all points in the canal network and a set of boundary

conditions on all physical boundaries of the solution domain. The initial concentrations are either all automatically set at the specified concentration of the receiving waterbody, or can be input on a cell-by-cell basis.

For each solid boundary (dead-end, bank, and bed) and for the air/sea interface at any time, the condition of zero mass flux is applied. One of the advantages of the model formulation is that the numerical method automatically satisfies this condition everywhere, except at the tidal entrances. Dispersion velocities, to satisfy this requirement, have been derived for the dispersion terms, which otherwise would require a condition to replace the spatial second derivative at the boundaries.

The tidal entrance boundary condition for the three-dimensional model is a revised form of the one-dimensional boundary condition. It is not identical because the circulation patterns produced by wind and salinity gradients may also cause flow reversals at the mouth of the canal, resulting in both inflow and outflow from different cells at the same time level.

In order to model this region of the network it has been assumed that, once the flow exits the system, it becomes well-mixed, and that the resulting concentration decays to the background as described for the one-dimensional model in Equation (7.14). In the model, a special cell is provided having a volume equal to that of the last segment in the tidal entrance reach. If the concentration in this cell is given by $\tilde{c}^{n}_{TE}$, and the volume by V^{n}_{TE}, then the resulting concentration at the end of a convective step, c^{n+1}_{TE}, is given by

$$\tilde{c}_{TE}^{n+1} = \frac{\sum_{j=1}^{NLAYY} \sum_{k=1}^{NLAYZ} V_{jk}^{n+1} c_{jk}^{n} + (V_{TE}^{n+1} - \sum_{j=1}^{NLAYY} \sum_{k=1}^{NLAYZ} V_{jk}^{n+1}) c_{TE}^{n}}{V_{TE}^{n+1}} \tag{7.86}$$

where

V_{jk}^{n+1} = volume of fluid transport out of reach from a cell, (L^3)

c_{jk}^{n} = concentration in above volume, (dimensionless)

and where the double summation occurs only over cells which have an outflow to the receiving waters. For the remaining cells, an inflow condition is defined such that when fluid of concentration c_{TE}^{n} enters the cell, the width of the volume in the cell becomes equal to the width of the cell, Δx_i.

$$c_{ijk}^{n+1} = R\tilde{c}_{ijk}^{n+1} + (1 - R)c_{TE}^{n} \tag{7.87}$$

where

R = width of volume in cell after convective step, (L)

$\tilde{c}_{ijk}^{n+1}$ = concentration of fluid in cell after convective step, (dimensionless)

Once the convective step is completed, and the uniform concentration is formed in the tidal entrance cell, its value is assumed to decay towards the background concentration, c_{RW}, described by

$$c_{TE}^{n+1} = c_{RW} + (c_{TE}^{n+1} - c_{RW}) \exp(-3\Delta t/\tau) \tag{7.88}$$

where

τ = tidal entrance decay coefficient, (1/T).

This condition was implemented in the three-dimensional model and was tested under a variety of conditions, including the case in which

only the tidal component of the velocity field operated on the flow. In all cases, the condition was found to work satisfactorily.

7.4 Model Accuracy, Stability and Convergence Criteria

The accuracy of a numerical model is often described in terms of the *order* of its accuracy, which is related to the number of Taylor-series terms included in the numerical approximation to the partial derivatives. Most common finite-difference and finite-element methods that have been used to model the mass-transport equation are first-order accurate because second and higher-order terms are not included in the numerical form of the forward-difference or the backward-difference expression for the partial derivative of the concentration. *Numerical dispersion* is a second-order error which occurs in first-order accurate numerical methods. The method of second moments, and the second-upwind differencing method with limited antidispersion and flux-corrected transport, are both second-order methods which do not produce numerical dispersion.

The order of accuracy is usually not a problem in most applications of the mass-transport equation, e.g. in rivers and estuaries, where the natural dispersion is very much larger than the numerical dispersion. However, in models of low energy tidal canals such as those along the southeast and south coast of the U.S., dispersion coefficients are very small and the numerical dispersion associated with a finite-difference or finite-element model can be greater than the natural dispersion, producing either unstable or severely attenuated results. Instabilities arise when the numerical dispersion becomes negative, and attenuation or damping occur with positive numerical dispersion. Often correction factors for the numerical dispersion are employed, but it is possible

that these factors can give rise to negative values of dispersion in portions of a solution, and hence instabilities.

The principal source of error in the method of second moments is computer round-off error, which is usually not significant except in the square-root operation. A square-root is used at each time step in each cell to calculate the width of the rectangular distribution in the cell. The results, however, are much more accurate than necessary considering the basic assumptions inherent in the model and the overall accuracy of the field data.

The stability of a numerical model may be considered in terms of the ability of the model to damp out small perturbations in the solution over time. These perturbations may be caused by errors due to the truncation of the infinite series approximations to the terms in the governing equation, or by round-off errors caused by the finite word length of the computer. For some numerical methods conventional stability analysis techniques may be used, but in most cases stability criteria can be developed that are based on the mechanics of the problem being investigated. The latter technique was used for the three-dimensional model, the resulting criteria being divided into a velocity criterion and a dispersion criterion.

7.4.1 Velocity Stability Criteria

The simplest velocity stability criterion is that pollutant mass should only be transferred between adjacent cells during one time step. If during a time step the velocity is too large, and mass is transferred from one cell through a second cell to a third, the square root term associated with the width of the distribution in a cell can become negative. This simple velocity criterion can be written

$$|u_{maxijk}| < \frac{\Delta x}{\Delta t} \tag{7.89}$$

where

u_{maxijk} = largest velocity expected in cell ijk, (L/T)

Since the velocity stability criterion applies in each cell of the model, it is necessary to consider the maximum expected velocity profile in each reach and junction in conjunction with its size in the direction of flow. For the case of the astronomical tide alone, the maximum velocity in each reach and junction can be expected at its downstream end since the tidal velocity is a function of distance from the dead-end. Also, since the vertical velocity distribution is assumed to be logarithmic and the transverse distribution parabolic, the maximum tidal velocity will be along the centerline in the surface layer. Assuming the length of each segment, Δx, is either constant or nearly constant, and considering the tidal component only, the critical cell is the center, surface cell at the downstream end of a reach or junction.

For wind-induced circulation superimposed on the tidal velocity distribution, the maximum velocity is still in the center in the surface layer. For density currents alone, induced by the movement of a saline wedge, the maximum velocities occur at the downstream end of each reach. However, because of the parabolic vertical velocity distribution within the wedge, the location of the critical cells at the downstream end of the reach may vary vertically between two or more cells. In this case, one cell in each vertical layer at the downstream end of each reach should be checked for meeting the velocity criterion.

The remaining two components of the velocity field are the bend-induced circulation and the lateral inflow. However, unless an undirectional flow, such as a river, is being simulated by means of the lateral

inflow capability, these two effects produce velocities that are small compared with the other velocity components. Thus, these components need not be considered in the stability analysis.

Usually the model will be used either with the astronomical tide only, with wind-induced tidal circulation, or with the wind-induced tidal circulation in the presence of a density wedge. These three cases will be examined separately. For a simulation with tide only, the maximum astronomical tidal component of the velocity field, u_{tmax}, from Equation (7.17) is

$$u_{tmax} = \left[\frac{1}{A}\frac{dv}{dt}\right]_{max} \left[\frac{d}{\Delta z}\,\ell n\left(\frac{d}{d - \Delta z}\right) + \frac{\ell n(29.73(d - \Delta z)/ke)}{\ell n(29.73d/ke)}\right] \tag{7.90}$$

The maximum wind-induced component expected, u_{wmax}, can be expressed as a function of the maximum component of the wind $|w_s \cos\theta|_{max}$ along the longitudinal axis of the reach

$$u_{wmax} = \frac{K_w\,|w_s \cos\theta|^2_{max}\,(d - \Delta z)^2}{4\,dN_z} \tag{7.91}$$

where

Δz = thickness of the top layer, (L)

The maximum wind-induced tidal velocity is therefore obtained by combining Equations (7.89), (7.90), and (7.91)

$$|u_{wmax}| + |u_{tmax}| < \frac{\Delta x}{\Delta t} \tag{7.92}$$

This form is based on the assumption that either the length of a section, Δx, is constant throughout the reach or that its variation is small (particularly if the variation causes Δx to decrease from its

value at the downstream end of the reach). If Δx does vary within a reach, it will be necessary to examine the criterion given by Equation (7.92) at other center-surface cells in the reach. Checking the stability criterion only at the tidal entrance is not sufficient if a wind-induced circulation is being simulated, since the criterion could be satisfied at the tidal entrance but violated at an upstream reach that is aligned more closely with the wind direction. It should be remembered that the wind-induced velocities are usually substantially greater than the tidal velocities, and care should be taken to check conditions carefully in all reaches in accordance with the above guidelines.

The vertical velocity profile within a saltwater wedge during the flood tide, before the wedge impinges on a dead-end, has been given by Equation (7.47). In the top layer of the saline wedge the maximum velocity, u_{smax}, is

$$u_{smax} = [\frac{u_4}{\Delta z} d_s \frac{dd_s}{dt}]_{max} [1 - (1 - \frac{\Delta z}{d_s})^3] \tag{7.93}$$

When a density current due to the motion of a saline wedge in a wind-induced tidal flow field is being simulated, the criterion for the maximum longitudinal velocity is

$$u_{max} = |u_{tmax}| + |u_{wmax}| + |u_{smax}| < \frac{\Delta t}{\Delta t} \tag{7.94}$$

This condition leads to a conservative criterion since worst case conditions for the wind-induced circulation and the density current ocurr in different cells. However, it is much simpler than a rigorous solution of the equation at each cell in the cross-section at the downstream end of each reach.

The methods outlined above apply only to reaches, because the wind-induced and density current features are not implemented in the junctions. To determine the effect of a given spatial increment in the junctions, Equation (7.89) can be used directly with the estimated velocities in cells adjoining the junction. However, this does not guarantee stable results because of the coupling of two different numerical techniques, the upwind differencing method in the junctions and the method of second moments in the reaches. Instabilities have been noted on occasion when stability criteria appeared to have been met; in all cases, stability was achieved by halving the time step.

7.4.2 Dispersion Stability Criteria

The basic dispersion criterion is that not more than half the difference in the mass of the substance can be transferred between adjacent cells during one time step. This can be written in terms of the dispersion coefficients

$$(E_x, E_y, E_z) \leq \frac{1}{2\Delta t} (\Delta x^2, \Delta y^2, \Delta z^2) \tag{7.95}$$

Assuming the background coefficient, E_o, is small compared to $(\Delta x^2/2\Delta t)$, a criterion can be developed for the dimensionless dispersion coefficient K_x from Equations (7.59) and (7.95)

$$K_x \leq 4.125\ \Delta x^2/\Delta t\ du_{max} \tag{7.96}$$

where u_{max} is given by Equation (7.94). The value of the right hand side of Equation (7.96) will usually be much larger than the value of K_x, which by Taylor's [1954] expression has values on the order of 0.1. Thus, in most cases this criterion can be assumed to be met.

Similarly, from Equations (7.60) and (7.95), the stability criterion for the lateral dimensionless dispersion coefficient is

$$K_y \leq 4.125\ \Delta y^2/\Delta t\ du_{max} \tag{7.97}$$

This latter condition, while more easily violated than the stability condition for coefficient K_x, is well satisfied in most practical cases since K_y is an order of magnitude less than K_x. Transverse stability will only be a problem in 3-dimensional simulations when there are many lateral divisions, and the time step, Δt, is large.

The criterion for the vertical dimensionless dispersion coefficient, K_z, is by far the most critical, and usually will be the one violated first in a simulation. From Equations (7.61), (7.63), and (7.95), and for E_o the same order of magnitude as $\Delta z^2/2\Delta t$, the stability criterion is

$$K_z \leq 9\ (\frac{\Delta z^2}{2\Delta t} - E_o)/d^2\ |\frac{du}{dz}|_{max} \tag{7.98}$$

$$\leq \text{(a positive value)}$$

As before, the vertical velocity profile is best evaluated by considering the cases of a wind-induced circulation with and without a density current. In both cases the tidal velocity component may be ignored to first order, since its effect is usually small and would automatically be satisfied for most practical selections of network cell structure.

Considering wind-induced circulation alone, Equation (7.30) may be differentiated to obtain

$$K_z \leq [\frac{\Delta z^2}{2\Delta t} - E_o]\ \frac{18\ N_z}{d^2\ K_w |w_s \cos\theta|^2_{max}} \tag{7.99}$$

which is conservative since it uses the surface value of du/dz.

In the case where the saltwater wedge is also present the condition is more complex, since it depends on the relative thicknesses of the fresh and saltwater layers. Taking a conservative approach for simplicity, it may be assumed that the velocity in the freshwater layer is the maximum velocity in the wedge multiplied by an inverse ratio of the thicknesses of the layers. Incorporating this assumption with the condition for the wind-induced circulation results in the expression

$$K_z \leq 9 \left(\frac{\Delta z^2}{2\Delta t} - E_o\right)/d^2 \left[\frac{K_w |w_s \cos\theta|^2_{max}}{2 N_z} + 3u_4 \left(\frac{dd_s}{dt}\right)_{max} \left(1 + \frac{d_s}{d - d_s}\right]\right. \tag{7.100}$$

The stability criteria given in this section provide guidelines for the selection of spatial and temporal increments for the numerical model. In most cases the spatial increments are chosen first, reflecting the desired detail that the user will want in the results. Once these values have been established, the time increment can be chosen to meet all the stability criteria. Once a particular model is operational, an examination of the calculated velocity field will show whether the criteria have been met and simpler forms of the criteria, such as Equation (7.89), may be used for this determination.

7.4.3 Convergence Criteria

A numerical technique is said to be convergent if the solution of the difference equation used to represent the theoretically-derived equation approaches the solution of the theoretical equation as the independent variables in the numerical approximations, Δx, Δy, Δz and Δt

approach zero [Smith, 1975].

For a particular numerical technique there must be a set of values for spatial and temporal increments which will provide results which are acceptably close to the exact solution. When spatial and/or temporal increments are too coarse the numerical solution will either rapidly diverge from the exact solution, or will achieve only relatively poor accuracy. When spatial and/or temporal increments are too small the cost of computing time becomes excessive.

Once the stability criteria have been satisfied, a numerical solution of the three-dimensional mass-transport equation will usually converge toward the exact solution. For this variable-coefficient equation it is difficult to establish the necessary and sufficient theoretical convergence criteria. It has been possible, however, to find stability guidelines by running several tests. The parameters of the 1600 ft long test canal are summarized in Table 7.3. A 5 mph wind toward the dead-end was simulated (Figure 7.11), with an initial concentration in the canal of 100 ppm. In all cases the results were depth-averaged.

The objective of the first test was to compare the convergence for various combinations of spatial and temporal increments. Over a range of 50 to 200 ft for Δx and 0.97 to 0.388 hr for Δt the results were within 5 to 6 percent after 50 tidal cycles. It is concluded from this test that once the stability criteria have been satisfied the model will yield accurate results.

The objective of the second test was to check the relative effect of changes in the number of vertical layers in simulating wind-induced circulation. For NLAYY equal to 1 and NLAYZ between 1 and 6 it was

found that the number of layers should be a multiple of three, and that the depth-averaged results are similar for three and for six layers, although the individual values varied somewhat from layer to layer in these two cases. It is concluded that three layers are sufficient in most cases, that six layers may be useful if more detailed vertical concentration profiles are required (and hydrodynamic and concentration data from the prototype canal are available in sufficient detail to justify using the additional resolution), and that more than six layers is definitely uneconomical.

The objective of the third test was to check the relative effect of changes in the number of lateral layers. For NLAYZ equal to 3 and NLAYY between 1 and 3 the results were virtually identical for the straight test canal. It is concluded that one lateral layer will usually be sufficient, except that when bends are included or the lateral distribution of concentration is desired, two or three lateral layers should be used. Using more than one lateral layer complicates the stability in a junction, since the longitudinal distance increment through the junction will be shortened to a fraction determined by the number of lateral layers in the branch canal which is connected at that location.

7.5 Model Verification With Field Data

In this section the results of simulations of four sets of field data will be summarized, both to calibrate the model and to check its accuracy. The first case is a study conducted by EPA [1975b] at Big Pine Key, Florida. Simulations by the EPA and Lee [1977] with one-dimensional models, and with CANNET3D, are compared and discussed. The second case is concerned with data from two field surveys by the

Hydraulic Laboratory in the 57 Acres Canal Network, in July and October, 1977. The portion of this canal network selected for the field work and simulation was found to be affected primarily by the astronomical tide and wind-induced circulation only. The third case is concerned with data from the Loxahatchee north canal, obtained by the Hydraulic Laboratory in June, 1977. This canal was influenced by a saline wedge as well as by wind and tide.

7.5.1 Case No. 1: Big Pine Key Canal III

Big Pine Key Canal III is one of several canals in the Florida Keys studied by EPA [1975b]. The canal (Figure 7.12) is almost 1600 ft long, 40 ft wide, and 10 ft deep at mid-tide. It has a rectangular, prismatic cross-section and is aligned with its longitudinal x-axis (defined as positive from the dead-end) predominantly North-Northwest.

On November 3, 1973, at high water slack, EPA released 500 ml of Rhodamine WT dye at mid-depth, 50 ft from the dead-end of the canal. At regular intervals after the release dye concentrations were sampled at mid-depth along the longitudinal centerline of the canal [EPA, 1975b, Figure 22, p. 45].

After the field survey EPA attempted to simulate the observed profiles using two available numerical models. First, the June 1973 version of the storm water management model (SWMM) [Langer, et al, 1971; Metcalf and Eddy, Inc., et al, 1971] was tried. This model does not incorporate a dispersion term in its formulation, and was found to be unsuccessful since the centroid of the dye cloud was not convected out of the canal. The Columbia River Model (CRM) [Callaway, et al, 1969] was then tried, with the hope that the presence of a dispersion term

would provide meaningful results. Although a computer simulation was not shown for Big Pine Key Canal III, one was performed and repeated for Big Pine Key Canal V, an identical adjacent canal to the West. Having found a measured longitudinal dispersion coefficient of 24 x 10^{-4} mi^2/day (0.77 ft^2/sec) at the midpoint of the canal, EPA managed to reproduce the measured profiles by *dividing* the dispersion term by five.

Lee (1977) also tried to model Big Pine Key Canal III using his one-dimensional model, which corrects for numerical dispersion. He used a linearly-varying longitudinal dispersion coefficient that matched the measured value of 26 x 10^{-4} mi/day (0.84 ft^2/sec) at mid-canal. He reproduced EPA's observed profiles by *increasing* the dispersion term by *two orders of magnitude* (Fig. 7.13).

From these results two conclusions can be drawn. First, the CRM suffers from severe numerical dispersion when used to model low-energy canals with small dispersion coefficients. This is predictable from the fact that both the CRM and SWMM were developed from the Dynamic Estuary Model (DEM) [Feigner and Harris, 1970] and therefore should have similar characteristics. The inclusion of a dispersion term cannot be expected to produce any better results when the principal source of error, numerical dispersion, is not corrected.

Secondly, EPA's observation concerning "the dye cloud centroid remaining in one place" [EPA, 1975b, p. 207] is to be expected in a model that includes only the tidal velocity field. A simple tidal prism calculation, using a typical tidal amplitude of 1 ft for the Big Pine Key area, indicates that maximum velocities of no greater than 0.01 fps can be expected, and then only at the tidal entrance.

In such a system, which is directly open to the ocean, and which

would not appear to be influenced by density currents, the principal forcing function must be the wind. To illustrate this point CANNET3 was used to simulate this case in two-dimensions. One lateral layer and three vertical layers were used, and the wind speed and direction were estimated from monthly averages for the Key West area. The parameters for this test are summarized in Table 7.4. The background dispersion coefficient, E_o, was set to zero since the wind speed had to be assumed to be constant, and thus its effect could be combined with that of K_z. The lateral coefficient in a two-dimensional model is zero, and it can be assumed that the longitudinal dispersion coefficient, K_x, has a very small effect compared to that of the vertical momentum transfer coefficient, N_z, and the vertical dispersion coefficient, K_z.

To calibrate the two latter coefficients, N_z was altered until the center of mass predicted by the model corresponded with its observed location after 12 hrs. K_z was then altered until the concentration values matched at the same time level. Figure 7.13 shows that good agreement was obtained by setting N_z to 0.005 ft^2/sec and K_z to 0.008 ft^2/ sec. As will be seen from the calibrations using the results of other field surveys, these values are very reasonable, considering also that no wind data were available from EPA's field survey. As the wind-induced velocity depends on the wind speed raised to the power 2.5, N_z would easily have a considerably higher value. Furthermore, dye decay data are not available for this example.

7.5.2 Case No. 2: 57 Acres Canal Network

Two comprehensive field surveys were conducted at the 57 acres canal site to obtain data for verification of the numerical model. While the

first survey in July, 1977, was essentially planned as a study of the feasibility of obtaining dye concentration profiles over successive tidal cycles for several days, it provided data which were useful in verifying the model. The second survey, in October 1977, provided a different set of data under different wind conditions, but many of the other aspects of the second study were the same as in the first. In this section only the aspects of the two field surveys which contributed to the verification of the model will be discussed. As was the case for the Big Pine Key Canal III simulation, the important calibration parameters are the vertical momentum transfer coefficient, N_z, and the dimensionless vertical dispersion coefficient, K_z. However, since the wind conditions in the two investigations at 57 Acres were different, the effect of the background dispersion coefficient, E_o, could also be studied. The analysis of the two surveys at the 57 Acres Canal site will therefore be developed simultaneously in this section, so that similarities and differences may be more easily compared.

During the first of the two comprehensive field surveys at the 57 Acres Canal site only salinity and temperature profiles, and longitudinal dye concentration profiles at successive high and low slack tides, were recorded (see Section 6.4.4.2 and Appendix B). For the second survey, measurements of vertical velocity profiles and wind velocity were added. The wind velocity recorder, took signals from sensors set at an elevation (10 m) above the trees and on the north side of the South Loop near the dead-end. For the July survey, a wind record was obtained from the nearest meteorological station (West Palm Beach Airport). From Figure 6.31 it can be seen that the recorded wind in October was fairly light, becoming calm at night, while for the July study the wind at West Palm

Beach was much stronger and more uniform, blowing from the East for much of the time (Appendix B, Figure B.3). During both studies a continuously recording tide gauage was set up at the dead-end of the South Loop and, as for the wind speed and direction data, the tidal elevations were digitized for input to the model.

For both studies 1500 ml of Rhodamine WT were released into the end section of the South Loop (Figure 7.14) at high tide. For the July experiment the dye was released as a point source at the surface and mixed with a paddle. For the October experiment the dye was injected into the top 3 ft layer under pressure and mixed across the canal. For about a one-half hour period before and after each successive high and low tide a longitudinal concentration profile was measured continuously along the South Loop and in as much of the rest of the network as dye could be detected, at a depth of 3 ft along the centerline.

After the dye concentration profiles from the field surveys had been reduced it was noted that there appeared to be a discrepency in the plotted concentration values. The first indication that there could have been an error was the fact that concentrations of the order of 100 ppb were measured one tidal cycle after the dye release in July, while concentrations on the order of 10 ppb were measured one-half and one cycle after the dye injection in October. After checking the data reduction procedure and finding no evidence of any procedural errors, a calculation of the concentration to be expected in the South Loop, assuming complete mixing, was made. For a 20 percent dye solution the concentration should be on the order of

$$c = \frac{5\ (\text{ml}) \times 0.2 \times 3.532 \times 10^{-5}\ \text{ft}^3/\text{ml})}{L \times W \times D} \tag{7.101}$$

where

L, W, D = length, width, and depth of a portion of the reach, (L) respectively, (L)

and therefore the resulting mean concentration for 1500 ml of this solution mixed uniformly to the first junction in the South Loop should be (from equation 7.101)

$$c = \frac{1500 \times 0.2 \times 3.532 \times 10^{-5}}{4200 \times 100 \times 8}$$

$$= 3.15 \times 10^{-9}$$

$$\cong 3 \text{ ppb}$$

Only if the mixing activity is so low that a thirtieth of the volume has dye mixed into it, will concentrations of 100 ppb be found. It was therefore expected that either the calibration procedure for the fluorometer, or the mixture of the standardizing solution, could be in error for the July data set.

When a dye study requires continuous sampling, by means of the high volume flow-through sampling arrangement on the Turner Designs fluorometer, a standard is prepared at the site in sufficient quantity to enable it to be circulated continuously through the fluorometer from a covered container as the fluorometer is calibrated. As explained in Section 6.2.9, this standard is prepared on site using a known quantity of 20 percent solution of Rhodamine WT and several gallons of canal water. The specific problem in preparation of the standard for the July study was to prepare 15 liters of 20 percent standard solution at 10^{-9} (1 ppb) concentration. Using a 10 ml graduated cylinder or a pipette, and a 1000 ml graduated cylinder, the mixture can be made in two or

three steps. However, it is admittedly somewhat difficult for a new member of the field crew to determine the steps to be accomplished, and then to execute these steps on site without assistance. This is considered to be the most likely source of error for the data discrepancy, but the verification method for the model was not dependent upon the absolute magnitudes of the concentration profiles and therefore this discrepency has not been considered to be of major significance.

For both test cases it was decided to simulate mass transport only in the South Loop, since it appeared that only a small portion of the dye escaped from this reach during the 40 to 50 hours of simulation (in both cases, and for computational economy. Thus the entrance to the South Loop was considered as a tidal entrance, and a high time rate of decay, τ, of 6.21 hours was specified because of the relatively small volume of dye escaping from the South Loop.

For the July study the 1500 ml of dye was released about 250 ft from the dead-end of the South Loop. Reach number 1 (Figure 7.13) was divided into 100 ft segments, with one lateral and three vertical layers. It was therefore assumed that, initially, the dye was well-mixed throughout the upper layer of the third segment from the dead-end, with an initial concentration in that cell of (from Equation 7.101)

$$c_i = \frac{1500 \times 0.2 \times 3.532 \times 10^{-5}}{100 \times 100 \times 3}$$

$$= 353.2 \text{ ppb}$$

The background concentration, c_{RW}, was estimated to be 20 ppb from field observations. The observed and computed longitudinal concentration profiles for reaches number 1 and 5 and junction number 2 are shown in Figure (7.15).

For the October study the 1500 ml of dye were injected into the canal a little farther away from the dead-end, as compared to the July study. From field observations the background concentration was found to be about 0.2 ppb. After the data had been reduced and plotted it was noted that there appeared to be a large amount of dye (peak value of 10 ppb) over a distance of about 800 to 1000 ft from the dead-end at the first low tide (Figure 7.16). There was in addition, a break in the data from 1000 to about 1500 ft, which had been extrapolated at the peak value in the comparison of computed and observed concentration profiles (Figure 7.17). This extrapolation is probably substantially in error. The cause for such a high concentration near the dead-end after a full tidal cycle cannot be deduced from the data. However, it should be realized that the circulation patterns near the dead-end of a canal, particularly one that does not have a simple, rectangular surface area, are quite complex in all directions, and it is likely that eddies are set-up that tend to trap pollutants for one or more tidal cycles. Realizing also that the dye concentrations are sampled at only one point in the cross-section, an apparent error of the kind shown in Figure 7.16 would seem to be possible. In fact, taking another viewpoint, it is somewhat surprising that the other longitudinal concentration profiles in the July data set are so consistent, considering that samples are only taken along a line approximately three feet in depth near the centerline.

It was decided not to begin the simulation of the October data at the time of the dye injection, but rather to begin with the digitized representation of the longitudinal concentration profile in reach number 1 at the first high tide after injection. The observed and resulting

computed longitudinal concentration profiles for reaches number 1 and 5 and junction number 2 are shown in Figure 7.17, in which time is measured from the low tide following injection.

The simulations of these sets of data met with both difficulties and success. It was difficult, for example, to accurately simulate the details of the July profiles, which was due at least in part to the lack of wind data at the site. However, as can be seen in Figure 7.15, the general movement of the dye cloud is fairly well reproduced.

The results of the simulations of the October data indicate that the center of mass is followed reasonably well from the first low tide after the beginning of the simulation (7 hours) to the final observation at the fourth low tide after the injection (38 hours) (Figure 7.17). One difficulty with this data set is the lack of fit of the profile after one-half tidal cycle (7 hours), which can probably be attributed to the use of the extrapolated concentration profile for the first low tide as the initial profile in the model.

For both cases fairly good agreement between simulated and observed locations of the center of mass was obtained with a value of 0.002 ft^2/sec for N_z. This result agrees quite closely with the value obtained for N_z (0.005 ft^2/sec) from the Big Pine Key Canal III study (Section 7.5.1), which used only an estimated wind. In fact, since the wind-induced velocity in the canal is related to the wind speed to the power 2.5 for moderate winds (Equations (7.22) and (727), the same value for N_z (0.002 ft^2/sec) could be used for both Big Pine Key Canal III and the 57 Acres South Loop if the wind used for the Big Pine Key Canal III simulation were multiplied by a factor of 0.7.

Once the transient locations of the center of mass of the dye

profiles at 57 Acres had been simulated, the next step was to reproduce the peak values of the profiles. Trial runs indicated that changes in the vertical dispersion coefficient, K_z, alone, would not be sufficient to match both sets of 57 Acres South Loop data. Since during the initial period of the October field survey, the winds were either very light or calm, it was decided that the background dispersion coefficient, E_o, could be used to match the peak concentration values for the first measured profile after the beginning of the simulation. This formed the basis of an iterative procedure on the values of E_o and K_z, which was continued until good agreement over the intial period of the two simulations was found. The values thus obtained were 0.0005 ft^2/sec for E_o and 0.0001 ft^2/sec for K_z.

With these values, and others given in Table 7.5, the simulations were then run to the end of the periods of record. The results have been shown in Figures 7.15 and 7.17. From these figures, it can be seen that reasonably good agreement was also found by matching the peaks, considering the difficulties inherent in obtaining *representative* data over a long period of time. However, it is recognized that the observations of the dye in the October data set from the South Loop at the 57 Acres site do not vary consistently with what would be expected considering mixing into the lower layers, loss through the junction to other parts of the canal network, and decay. Most of the disagreement can probably be attributed to the method of sampling dye concentrations and, possibly, the recalibration of the fluorometer at the beginning of each slack tide measurement.

After consideration of the above, a more useful set of data could probably be obtained by sampling at several points in the cross-section

at discrete locations in the canal. For example, dye concentration samples at at least two stations laterally (centerline and half the distance to one bank) and three stations vertically (one in each layer) at perhaps ten cross-sections would probably require 15 to 30 minutes per cross section (including travel and mooring time) or 2-1/2 to 5 hours, depending on the efficiency of the sampling system. Two full crews could reduce this sampling period to a reasonable value. However, as explained in Chapter 6, it is not feasible to require one field crew to cover every, or even every other, slack tide profile over an extended period of time.

In the South Loop, at station 3 (Figure 6.24), there is a sharp bend. This feature was not included in the simulations described above because of the inconsistencies in the data and the fact that the concentrations were only sampled at a depth of 3 ft at the centerline. During the October field survey, however, three velocity towers were setup just downstream of the bend to obtain data for testing the accuracy of the bend simulation. This bend was included for a test with the October concentration data, using three vertical and three lateral layers. A predicted and measured velocity field were shown in Figure 7.4, where the theory of secondary currents was discussed, which showed that field conditions are well modeled in bends. From this same simulation the vertical dispersion coefficients between the top and bottom layers, E_z, which are a function of the local vertical velocity gradient, are plotted with the component of wind speed along reach number 1 in Figure 7.18. From this figure the importance of a variable vertical dispersion based on the local velocity gradient can be readily seen. Most models simply provide a constant E_z. However, this could lead to erroneous

results in simulating low energy tidal canals in which flow reversals are common. In the reversal layer substantial mixing, not predicted by a constant vertical dispersion coefficient, is expected and can only be modeled using a form of E_z based on the value of the vertical velocity gradient at that location.

In concluding this discussion of the verification of the model with data from the July and October 57 Acres field surveys, good agreement between observations and calculated concentration profiles has been obtained for values of 0.002 ft^2/sec for N_z, 0.0005 ft^2/sec for E_o, and 0.0001 ft^2/sec for K_z, in spite of some data inconsistencies. The values of N_z and K_z compare favorably with the results of the Big Pine Key Canal III test. A sample test run from the computer for the October simulation may be found in Morris, Walton, and Christensen [1978, Appendix G].

7.5.3 Case No. 3: Loxahatchee North Canal

The Loxahatchee River site (Figure 6.25) consists of two parallel canals, the North and South Canals, which are 2200 ft long; 70 ft wide and 7 ft deep at mid-tide. The alignment angle of the canals is approximately 270 degrees (west). During a field investigation by the Hydraulic Laboratory in June, 1977 (discussed in Section 6.4.4.1), 75 ml of Rhodamine WT dye were injected and mixed in the upper half depth, across the width, 200 ft. from the dead-end of the North Canal, 3 hours before low tide. Sampling was performed at regular intervals at discrete points in the vertical and along the longitudinal centerline of the canal, as indicated by the sample readings in Figure 6.28.

During the field investigation, several wind speed readings were taken using a hand-held meter. However, because of the infrequency of

the readings, and because of the difficulty in obtaining an accurate direction, wind data collected at the closest meteorological station, West Palm Beach Airport, were used for simulation purposes. This record was digitized, together with the tidal elevations obtained from a continuous tide recorder located near the mouth of the canal, as input to the model. The tidal entrance decay coefficient, τ, was assigned the value 1, as the receiving waterbody was large and good mixing could be anticipated.

Measured salinity profiles (Figure 6.26) indicate that this is a system which is influenced by the movement of a saline wedge entering the canal during the flood tide, and receding during the ebb tide. Thus, it was decided to analyse this system by including the saline wedge capability of the model, as coded in the subroutine WEDGE. As discussed in Section 7.3.2, modeling of a saline wedge requires that the simulation begin at low tide. Unfortunatley, the dye was injected into the system 3 hours before low tide. However, the first set of measurements were taken during the following low water slack tide, and therefore these readings were digitized as the initial concentration profile in the computational cells of the model. The background concentration, c_{RW}, was measured to be 0.03 ppb.

The canal was divided into one lateral layer and three vertical layers, and the simulation proceeded for 50 hours. This test run was used to study the salt wedge parameter, u_4, introduced in Equation (7.43). The remaining parameters, N_z, K_x, K_y, K_z, and E_o, assumed the values chosen for the 57 Acres simulation runs. The values of the various parameters are listed in Table 7.6.

From Figure 7.19, it can be seen that good agreement was found

between observed and computed concentration values for a value of 3000 for u_4. Also, the computed vertical velocity profile, as shown in Figure 7.6, is a good match for the observed profile measured at the corresponding time.

As stated before, only one canal exhibiting these salt wedge conditions was studied, and although it is fairly typical in geometry to other canals influenced by density currents, the value of u_4 may have a variation which is dependent on the length of the canal. Thus, it is recommended that before a simulation of a proposed canal network is begun, it would be advisable to first check the value of this parameter on a canal of similar length, to determine both u_4 and its dependence on the length of a reach.

The verification of the model with the Loxahatchee north canal data showed that the values for the diffusion, dispersion, and vertical momentum transfer coefficients, obtained originally by fitting the model to the 57 Acres data, is representative of at least two different canals on the east coast of Florida. It remains to be seen how these coefficients vary in other locations.

7.6 Application of the Model

In contrast to many other two- and three-dimensional numerical models, CANNET3D is simple to set-up provided the proper field measurements are available. A user's manual, prepared by Walton [in Morris, Walton, and Christensen, 1978, Appendix D] provides an annotated list of the input variables and their formats in the order required for input to the computer. Setting up the model for a simulation requires decisions or values for variables in five different categories:

1. system-wide specifications
2. canal network layout and geometry
3. lateral inflows and cell decay coefficients
4. initial and boundary conditions
5. adjustments for stability

The system-wide specifications consist of the tidal amplitude and tidal period (if constant), the concentration in the receiving water-body, the spatial and temporal increments (including the output printing and plotting intervals and scale values), the total number of tidal cycles and the five diffusion coefficients.

The canal network layout is specified by the numbers of reaches, junctions, lakes, tidal entrances, and the numbers of layers vertically and laterally. The geometry is specified by the locations (with respect to junctions), orientations, lengths and cross-sectional dimensions of reaches, and the locations (with respect to reaches), depths and inverse side slopes of the junctions. If reaches with bends are to be simulated, the bend data for each reach also will be specified. The number of sections in each reach, and the widths and lengths of cells in the junctions, which at first appear to be geometrical variables, actually do not affect the geometry of the network and are adjusted to assure that numerical stability is achieved.

The cell decay coefficients and lateral inflows are specified on a cell-by-cell basis. The cell decay coefficients are treated as constants throughout the period of the simulation. The lateral inflows consist of a value for unit flow, q_I, and its concentration, c_I, which can be set either as a constant or can be read-in at each time interval at which tide and/or wind data are specified. In the latter case, intermediate

values will be interpolated automatically. This interpolation is specified by the two variables WTIME, which is the period in hours over which the moving average is to be calculated, and INTERP, which is the number of time steps between reading new input values.

The initial and boundary conditions consist of the initial concentrations in each cell, the tidal entrance decay coefficient, the four salt-wedge coefficients, and the tide and/or wind data if these values are variable during the simulation. The initial concentrations can only be read-in on a cell-by-cell basis in the current version of the model, since there are many possible combinations of inflow configurations. The tidal entrance and salt-wedge coefficients are constant for the entire simulation. The tide and/or wind data are read in at intervals specified by the variable INTERP.

Adjustments for stability are made through the time increment, DTH, and the division of reaches and junctions into sections or cells. Assuming that any reasonable subdivision of a reach results in a stable simulation, it is often convenient to divide the reach into that number of equal-length divisions which will provide outputs at convenient plotting distances. Alternatively, the lengths of each division within a reach can be read into the program individually. The lengths of the cells in a junction, the number of which are automatically determined by the number of vertical and lateral layers in the model, are calculated in the program from the specified total longitudinal and lateral lengths, DXJN and DYJN, of the junction. These total lengths can be increased if necessary to achieve model stability.

Examples of the application of the model to some relatively simple canal networks are described in Chapters 8 and 9. In some cases, such

as the simulations beginning with high concentrations in all cells in the network, and those with constant lateral inflow in the surface layers, the program was modified for ease of inputting these values. Provisions for these kinds of boundary conditions will be provided in the future.

Table 7.1 - Typical Measured Canal Parameters.

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

Table 7.2 - Comparison Between Horizontal Water Surface Assumption and Harleman and Lee's Hydrodynamics Model.

Fraction of tidal cycle, t/T	Deviation from time mean depth ft HWSA	Deviation from time mean depth ft HLM^2	Water surface slope at x, $\partial d/\partial x(x)$ HLM^2
	(a) At x = 500 ft		
0	-2.00000	-2.00141	-1.41×10^{-6}
0.25	-0.00001	-0.00265	-2.64×10^{-6}
0.50	2.00000	2.00092	9.20×10^{-7}
0.75	0.00548	0.00896	3.48×10^{-6}
	(b) At x = 4,500 ft		
0	-2.00472	-2.00551	-7.9×10^{-7}
0.25	-0.00729	-0.00800	-7.1×10^{-7}
0.50	2.00309	2.00360	5.1×10^{-7}
0.75	0.01557	0.01675	1.2×10^{-7}
	(c) At x = 9,500 ft		
0	-2.00693	-2.00709	-1.6×10^{-7}
0.25	-0.00867	-0.00866	1.0×10^{-8}
0.50	2.00453	2.00463	1.0×10^{-7}
0.75	0.01829	0.01837	8.0×10^{-8}

Note: 1. HWSA = horizontal water surface assumption.
2. HLM = Harleman and Lee's model [1969].

Table 7.3 - Constant Parameters for Three-Dimensional Model Test Canal.

PARAMETER	VALUE
Length, L	1600 ft
Bottom width, b	40 ft
Mean tidal depth, d_o	10 ft
Inverse side slope, left bank, s_L	0
right bank, s_R	0
Tidal amplitude, a	0.5 ft
Nikuradse's equivalent sand roughness, k	5 ft
Reach alignment angle, RANG	0 degrees
Reach decay coefficient, RDECAY	0/hr
Dimensionless dispersion coefficient	
longitudinal, K_x	0.1
lateral, K_y	0.001
vertical, K_z	0.05
Background dispersion coefficient, E_o	0 ft^2/sec
Vertical momentum transfer coefficient, N_z	0.01 ft^2/sec
Initial concentrations in reach, c_i	100 ppm
Background concentration, c_{RW}	5 ppm
Constant wind speed, w_s	5 ppm
Constant wind direction	0 degrees

Table 7.4 - Parameters for Big Pine Key Canal III, Case No. 1.

PARAMETER	VALUE
Length, L	1600 ft
Bottom width, b	40 ft
Mean tidal depth, d_o	10 ft
Inverse side slope, left bank, s_L	0
right bank, s_R	0
Tidal amplitude, a	0.5 ft
Nikuradse's equivalent sand roughness, k	5 ft
Reach alignment angle, RANG	0 degrees
Reach decay coefficient, RDECAY	0/hr
Dimensionless dispersion coefficient	
longitudinal, K_x	0.1
lateral, K_y	0.001
vertical, K_z	0.008
Background dispersion coefficient, E_o	0 ft^2/sec
Vertical momentum transfer coefficient, N_z	.005 ft^2/sec
Background concentration, c_{RW}	0 ppm
Constant wind speed, w_s	5 mph
Constant wind direction	0 degrees

Table 7.5 - Parameters for 57 Acres, Case No. 2.

PARAMETER	REACH 1	REACH 2	REACH 3	REACH 4	REACH 5
Length, L (ft)	4200	1300	700	900	2500
Bottom width, b (ft)	65	50	50	60	70
Mean tidal depth, d_o (ft)	8.0	6.2	6.3	7.7	8.0
Inverse side slope: left bank, s_L	3	3	3	3	2
right bank, s_R	3	3	3	3	4
Reach alignment angle, RANG	240	340	110	110	30
Equivalent sand roughness, k (ft)	15	15	15	15	15
Reach decay coefficient, RDECAY (1/hr)	0.0	0.0	0.0	0.0	0.0
Number of upstream junction	1	1	1	3	2
Number of downstream junction	2	3	3	2	4

Table 7.5 - continued.

PARAMETER	VALUE
Tidal amplitude, a (ft)	1.15
Dimensionless dispersion coefficient	
longitudinal, K_x	0.1
lateral, K_y	0.01
vertical, K_z	0.0001
Background dispersion coefficient, E_o (ft^2/sec)	0.0005
Vertical momentum transfer coefficient, N_z (ft^2/sec)	0.002
Background concentration, c_{RW} (ppm): July, 1977	6
October, 1977	0.2
Number of layers: lateral, NLAYY	1
vertical, NLAYZ	3

Table 7.6 - Parameters for Loxahatchee North Canal, Case No. 3.

PARAMETER	VALUE
Length, L	2000 ft
Bottom width, b	50 ft
Mean tidal depth, d_o	7 ft
Inverse side slope, left bank, s_L	2
right bank, s_R	2
Tidal amplitude, a	1 ft
Nikuradse's equivalent sand roughness, k	10 ft
Reach alignment angle, RANGE	270 degrees
Reach decay coefficient, RDECAY	0/hr
Dimensionless dispersion coefficient	
longitudinal, K_x	0.1
lateral, K_y	0.01
vertical, K_z	0.0001
Background dispersion coefficient, E_o	0.0005 ft^2/sec
Vertical momentum transfer coefficient, N_z	0.002 ft^2/sec
Background concentration, c_{RW}	0.03 ppb
Depth to saltwater interface, DSM	3 ft
Coefficient of Equation (7.43), u_4	3000
Density of freshwater, RHOF	1.94 slugs/cu ft
Density of saltwater, RHOS	1.99 slugs/cu ft

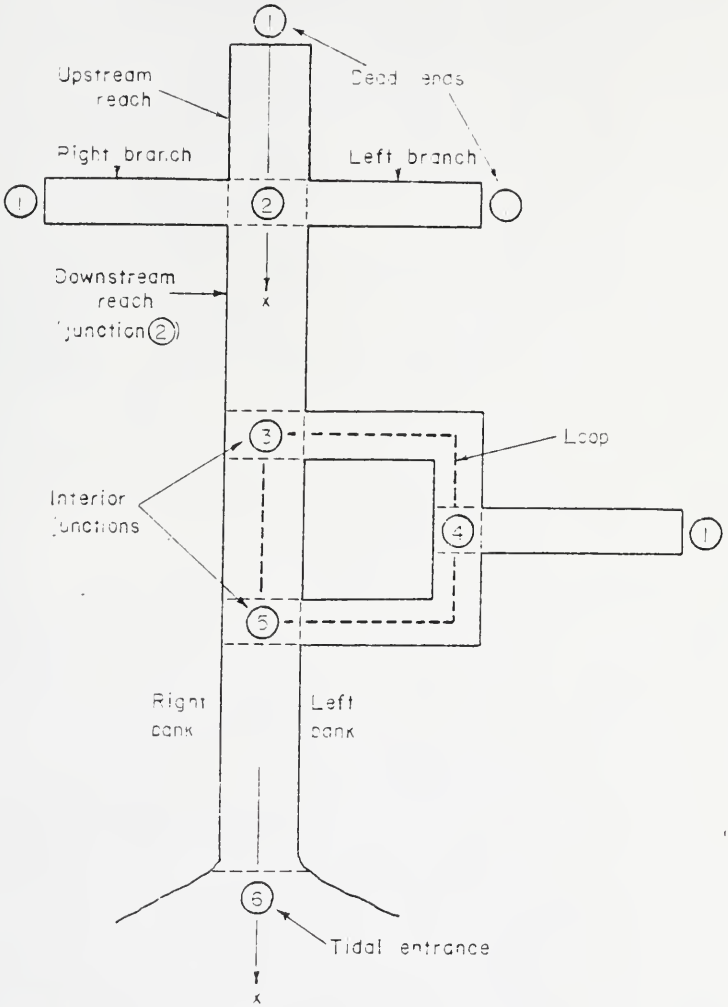


Figure 7.1 - Definition Drawing of Canal Network.

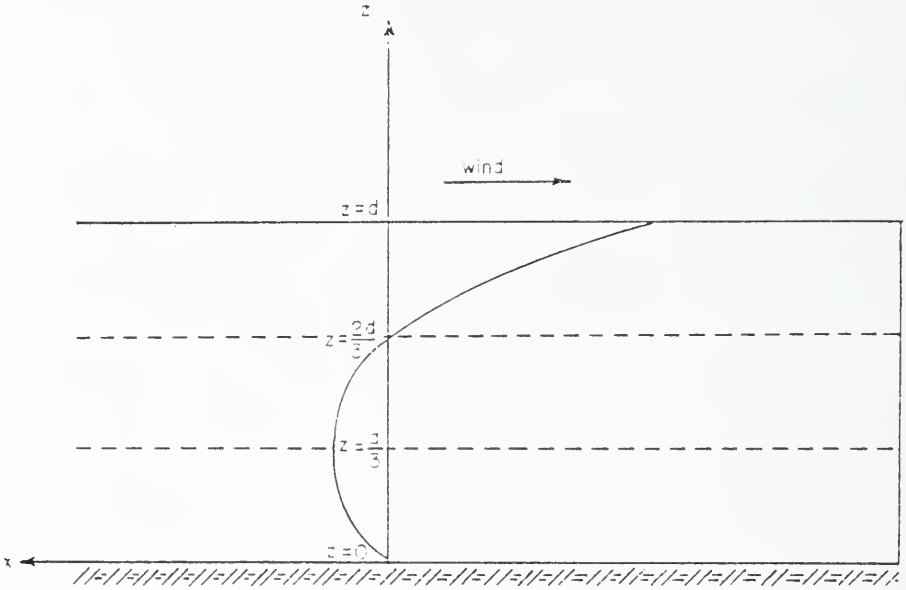


Figure 7.2 - Theoretical Wind-Induced Vertical Velocity Profile.

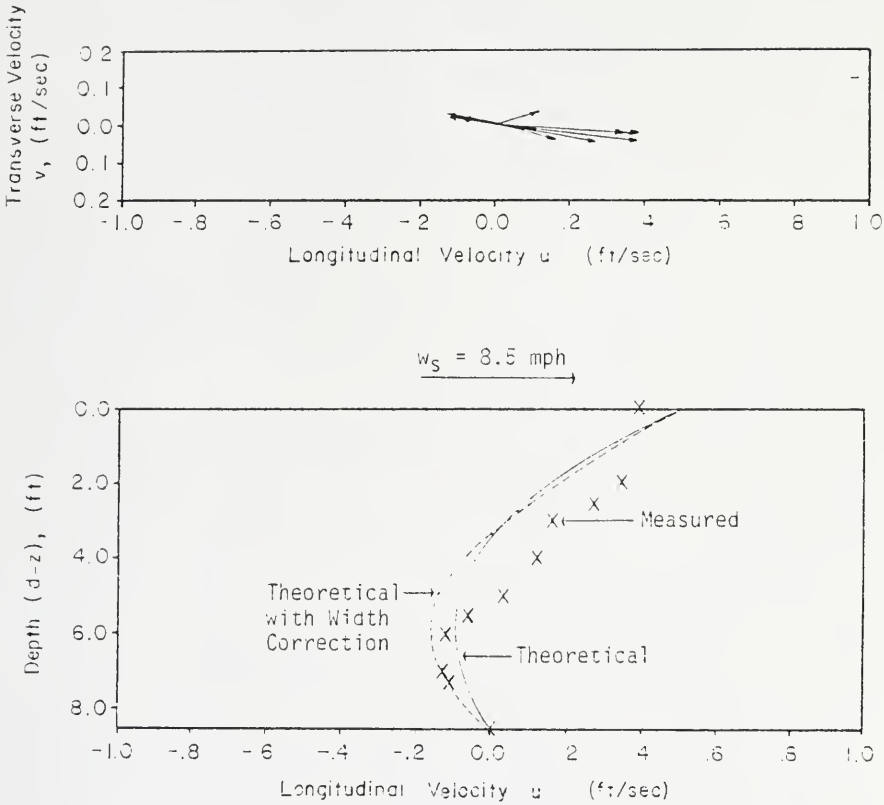


Figure 7.3 - Comparison Between Observed and Theoretical Wind-Induced Vertical Velocity Profiles, With and Without Width Correction (N_z = 0.002 ft^2/sec) - 57 Acres.

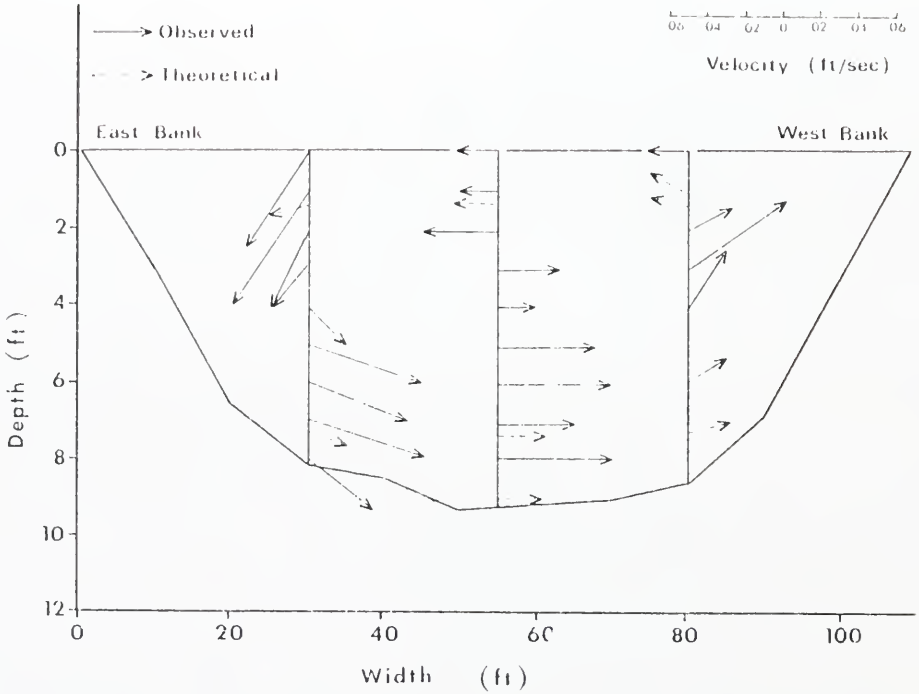


Figure 7.4 - Comparison Between Observed and Computed Lateral Velocities Induced by Bend in South Loop of 57 Acres System.

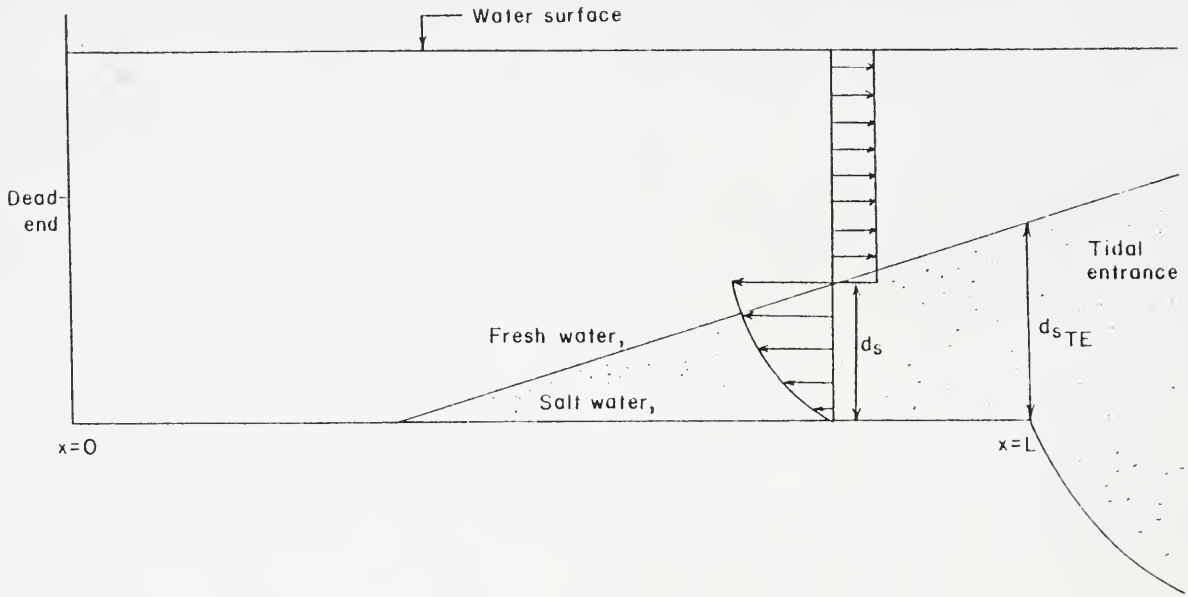


Figure 7.5 - Schematic Drawing of a Salt Wedge Entering a Canal Showing Definitions.

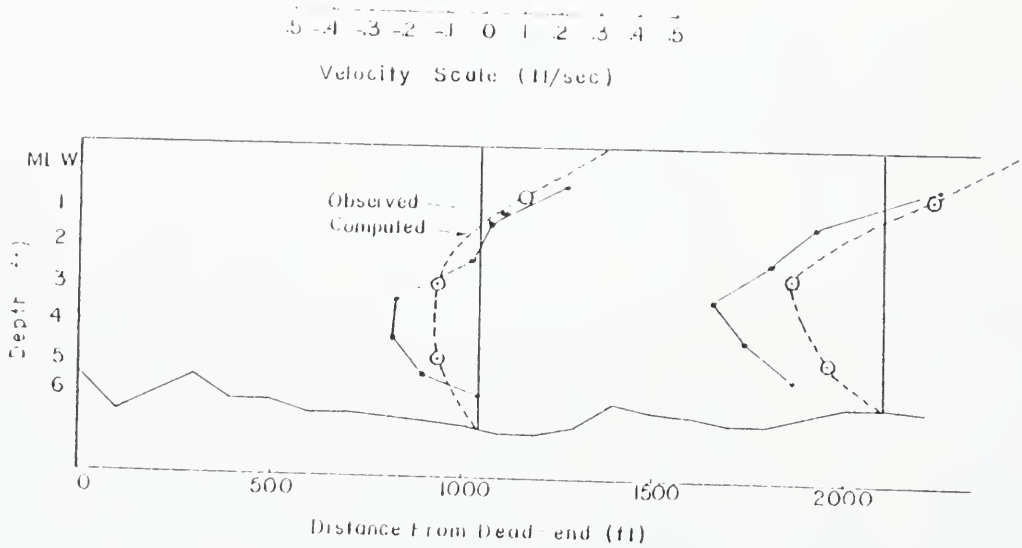


Figure 7.6 - Comparison of Observed and Computed Velocity Profiles for Loxahatchee River Site.

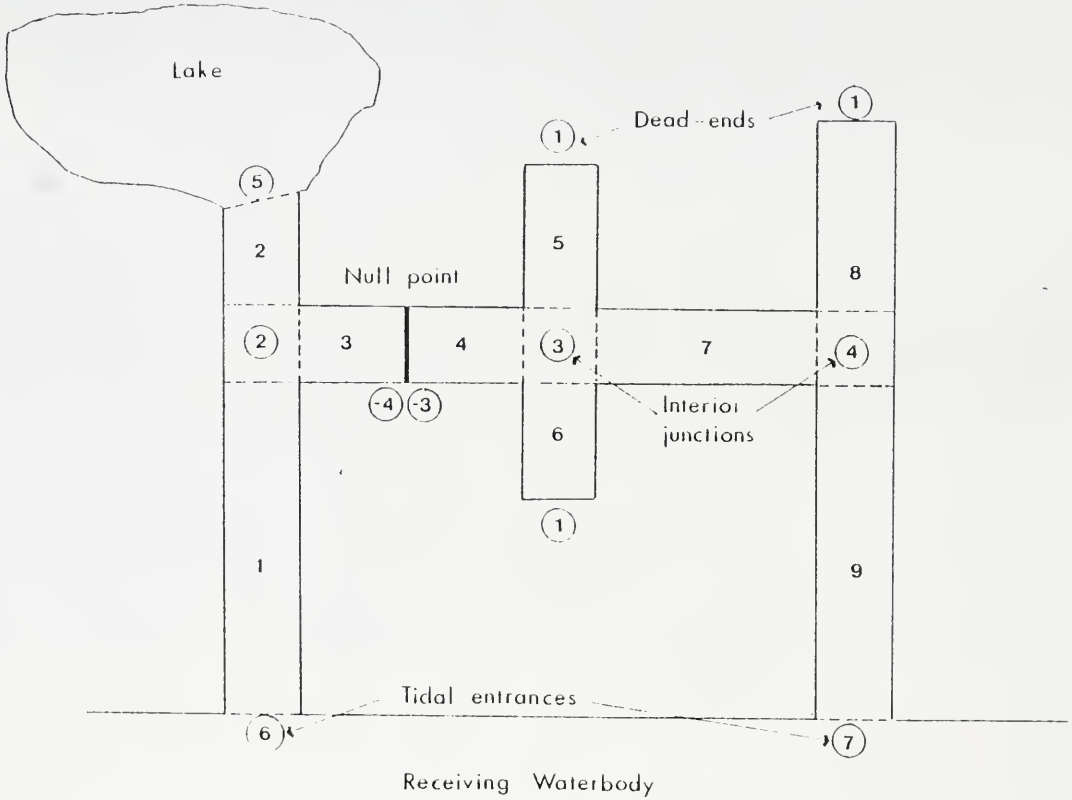


Figure 7.7 - Schematic Layout of Canal Network Showing Features.

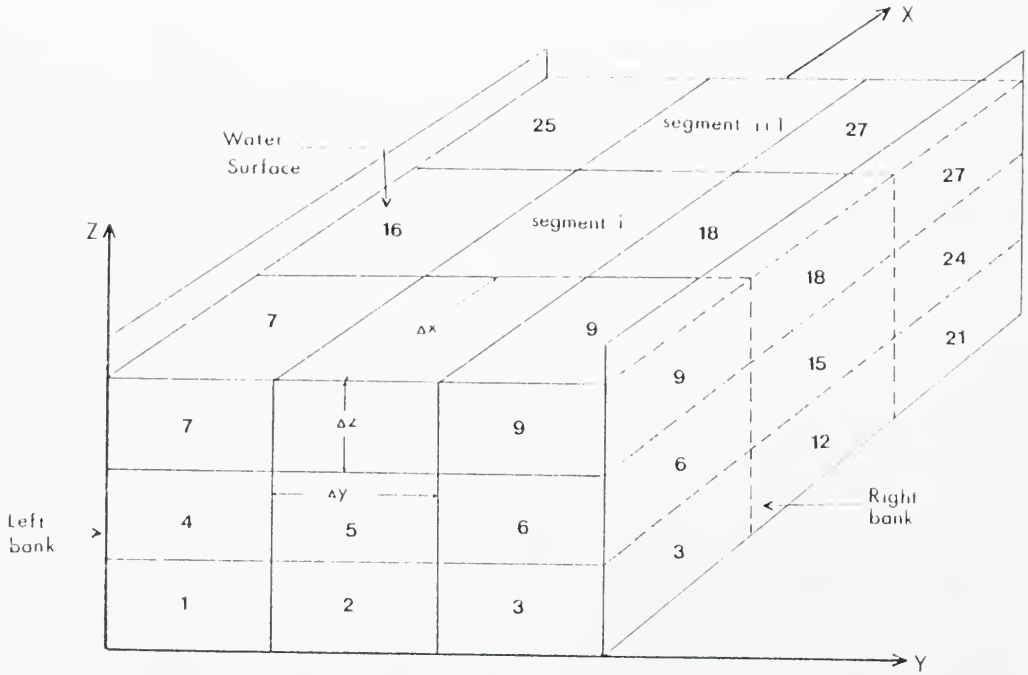


Figure 7.8 - Cell Structure in Reach.

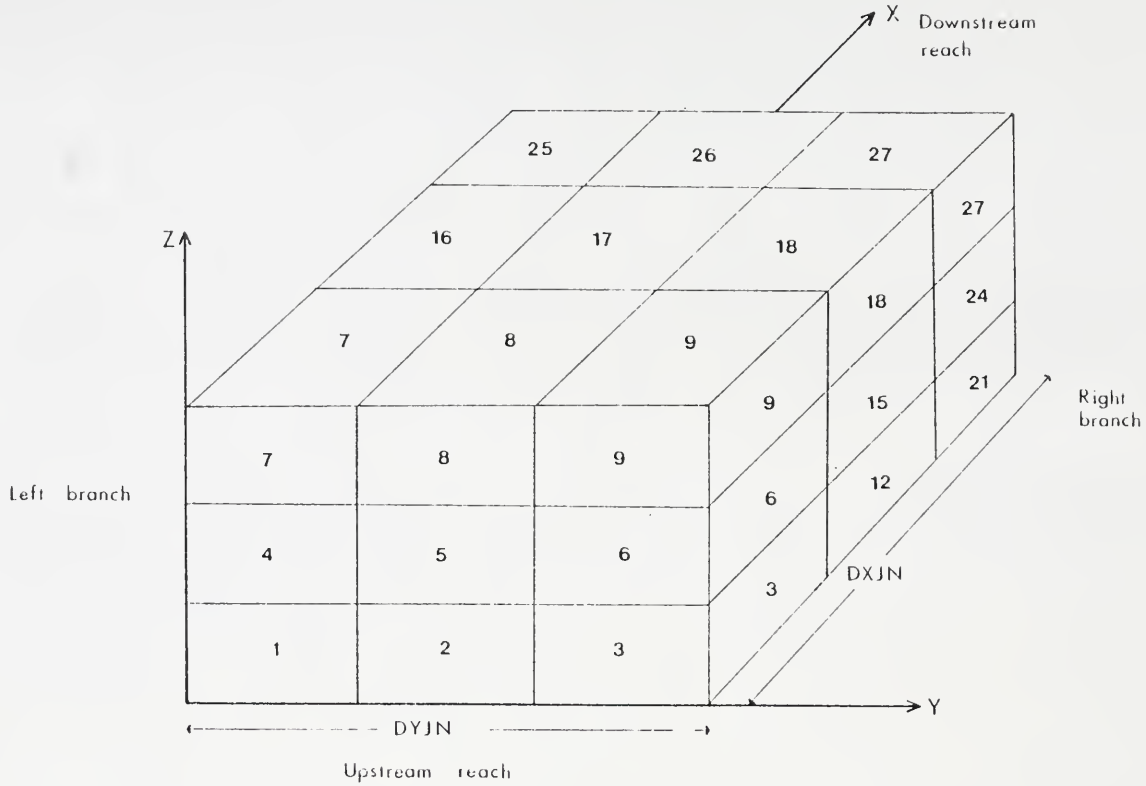


Figure 7.9 - Cell Structure in Junction.

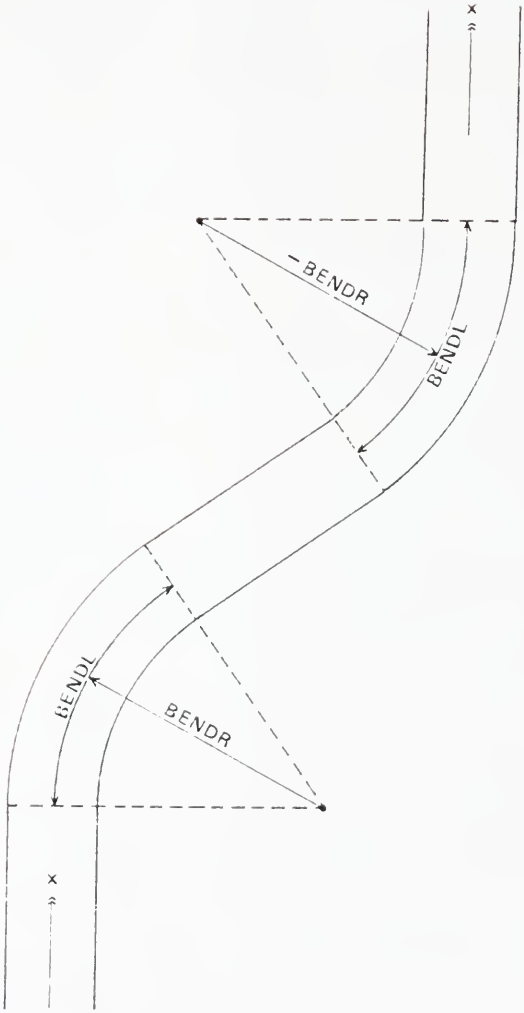


Figure 7.10 - Schematic Layout of Bend.

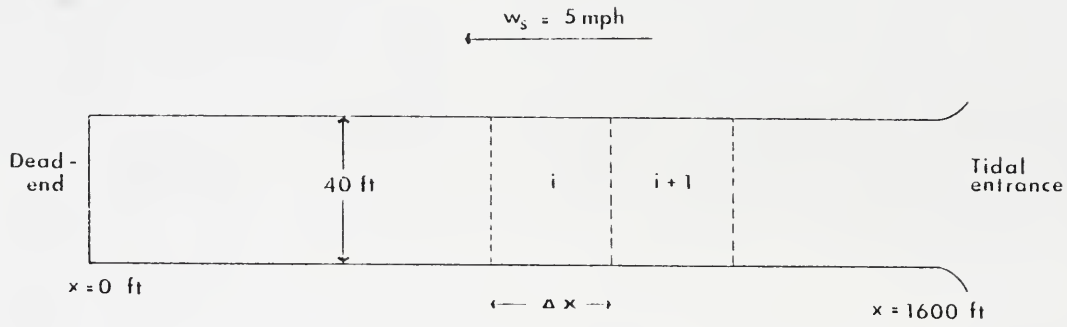


Figure 7.11 - Schematical Canal for Three-Dimensional Model Tests.

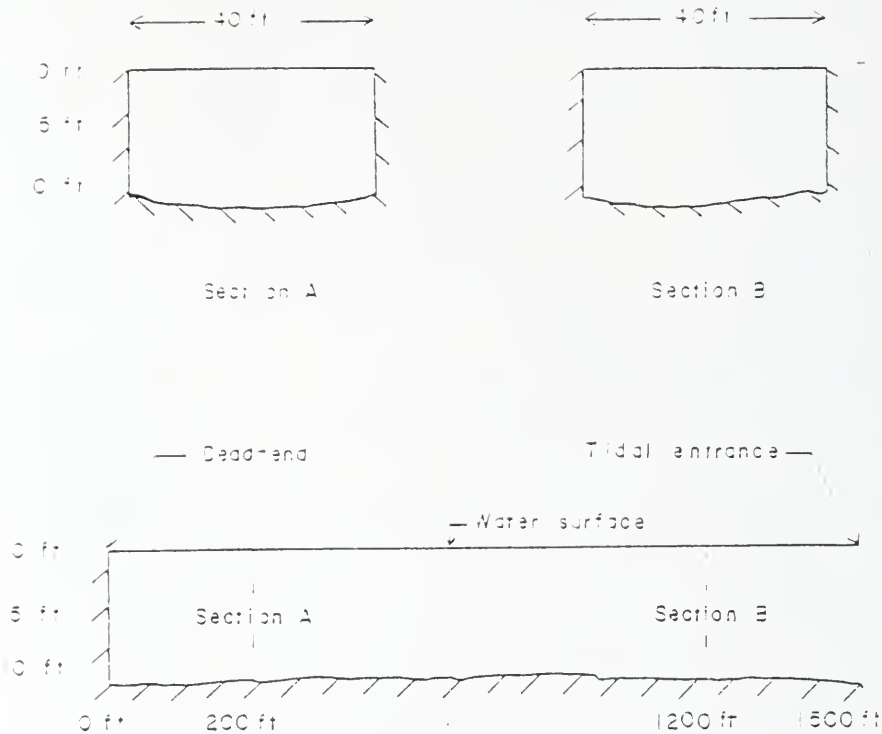


Figure 7.12 - Typical Longitudinal Sections and Cross-Sections, Big Pine Key Canal III, FL [EPA, 1975b, 1975].

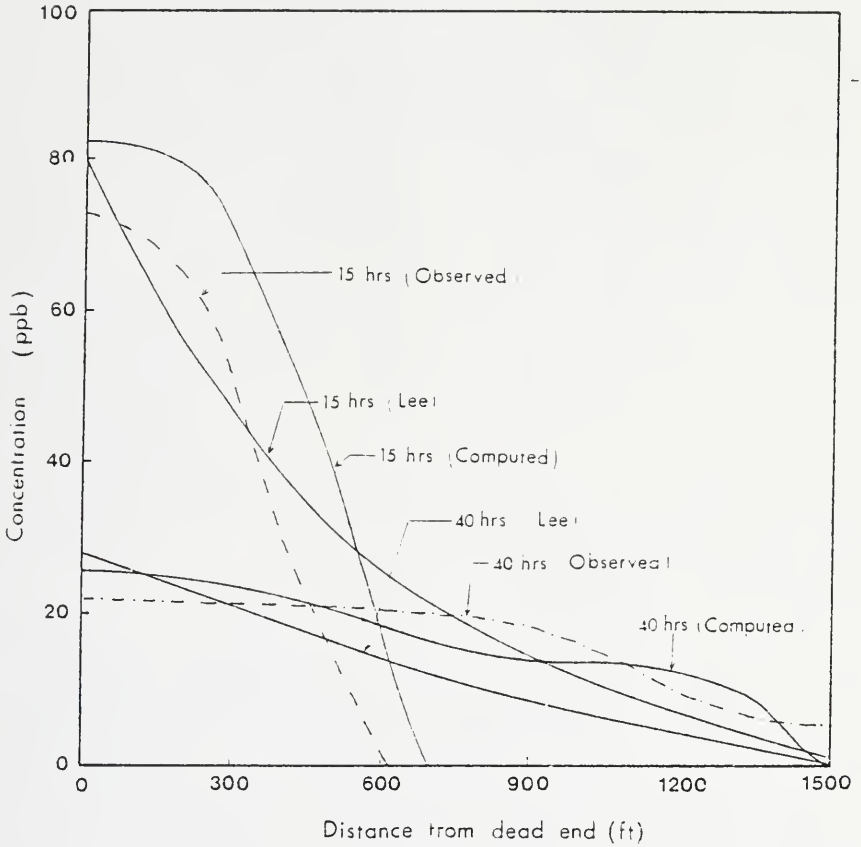


Figure 7.13 - Case No. 1: Observed and Predicted Concentration Profiles for Big Pine Key Canal III.

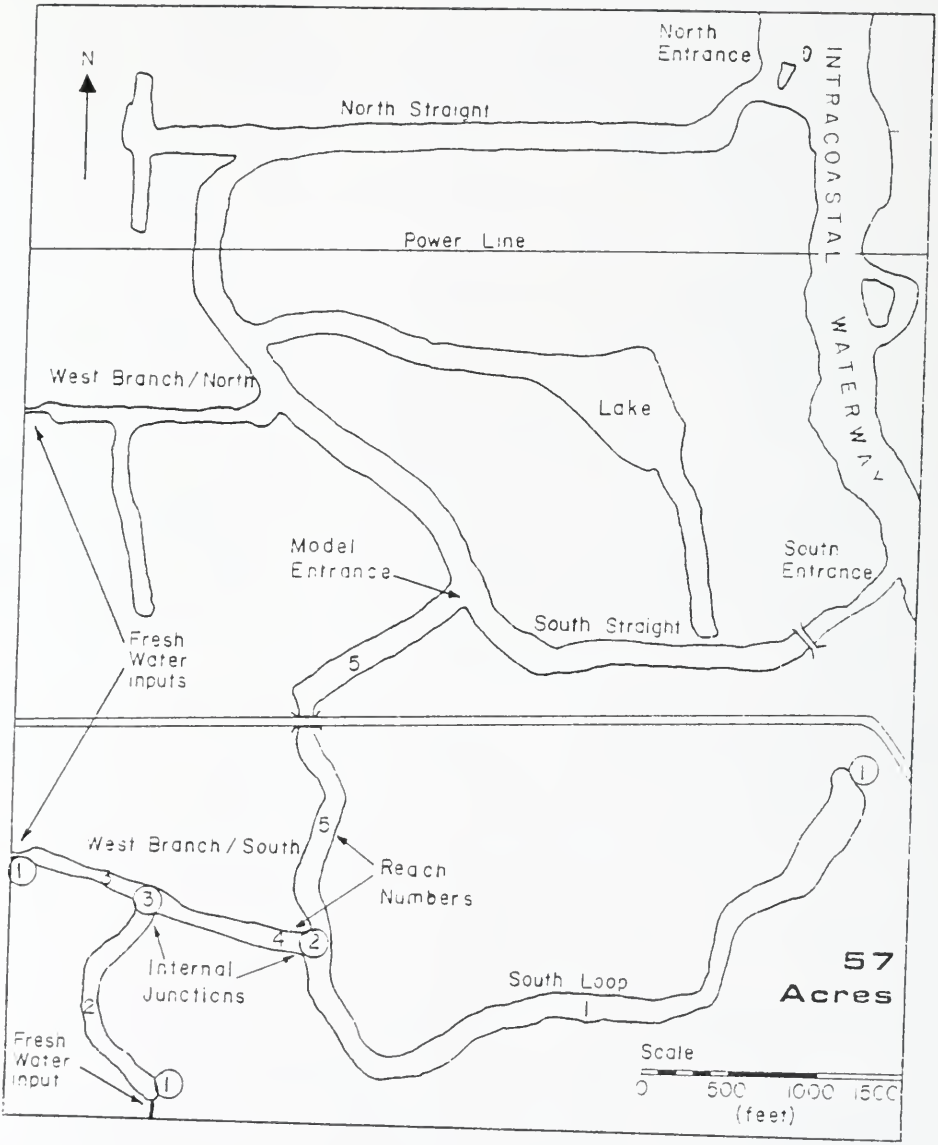


Figure 7.14 - Model Layout of Reaches and Junctions of 57 Acres System.

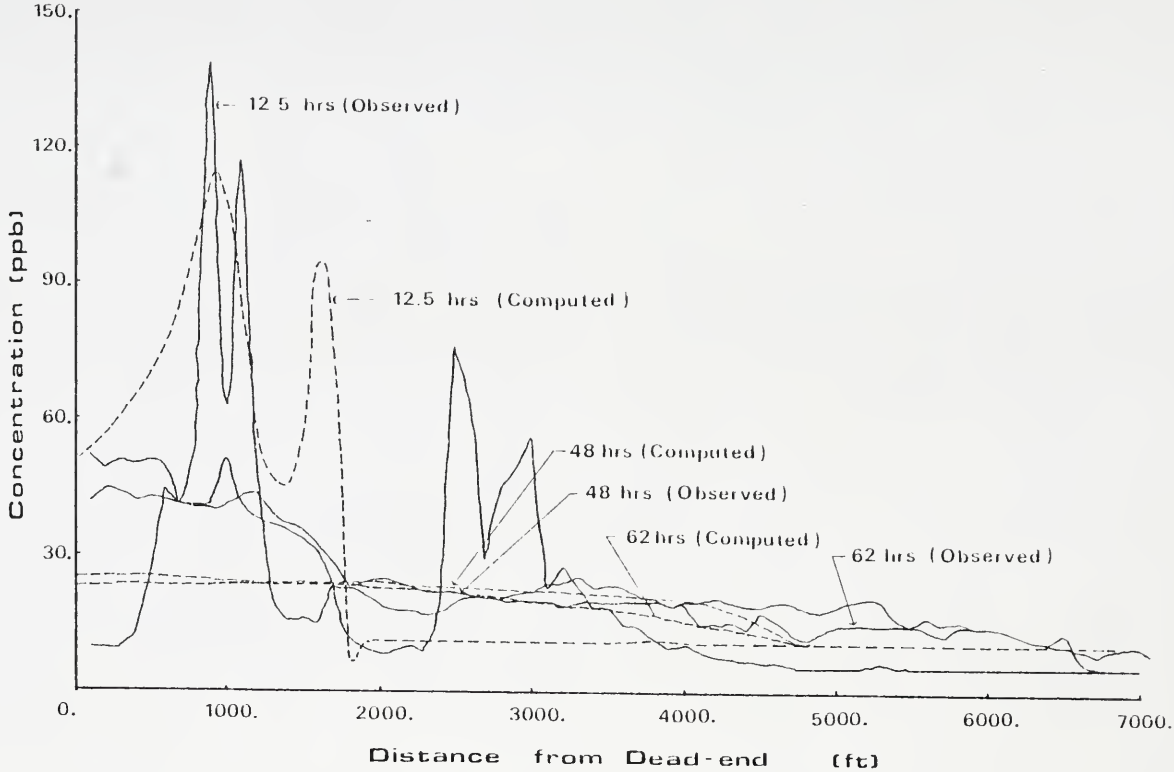
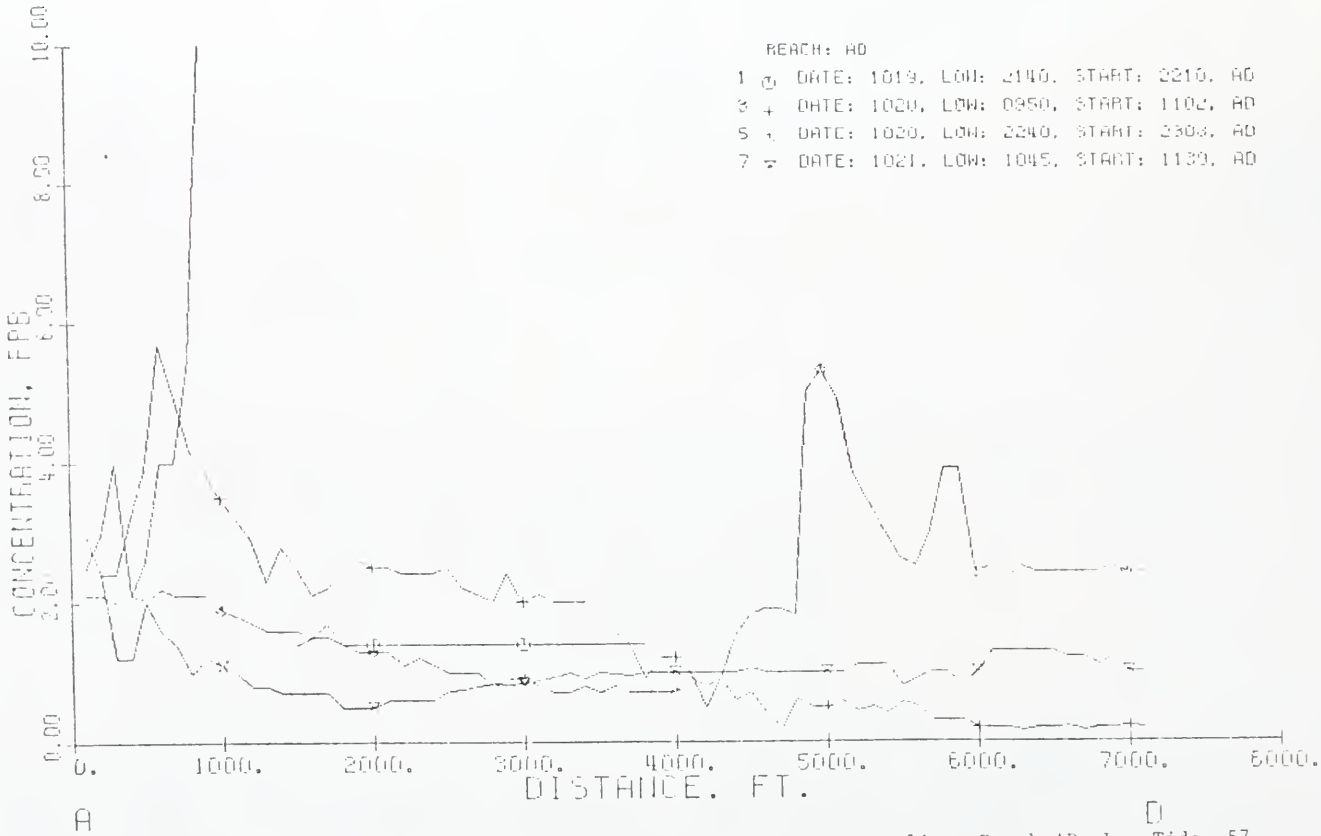


Figure 7.15 - Case No. 2: Observed and Predicted Concentration Profiles for 57 Acres, July, 1977.



A

D

Figure 7.16 - Measured Dye Concentration, at 3 ft Depth Along Centerline, Reach AD, Low Tide, 57 Acres Canal System, October, 1977.

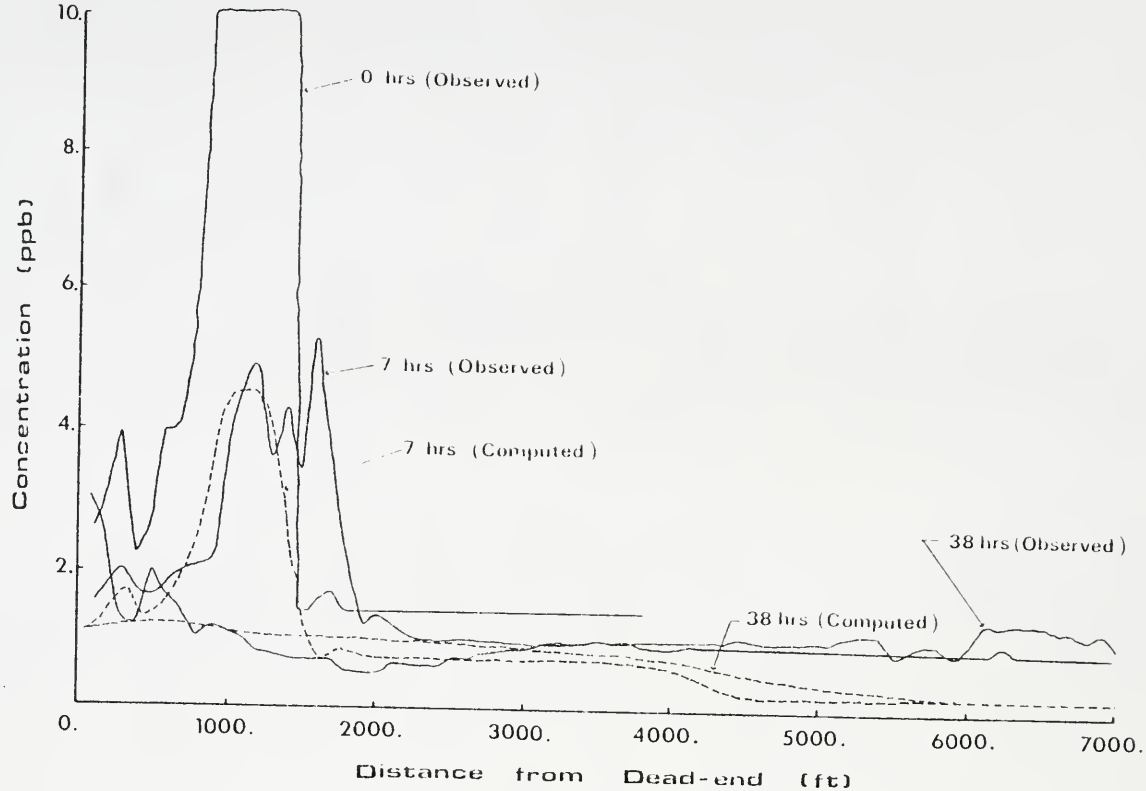


Figure 7.17 - Case No. 2: Observed and Predicted Concentration Profiles for 57 Acres October, 1977.

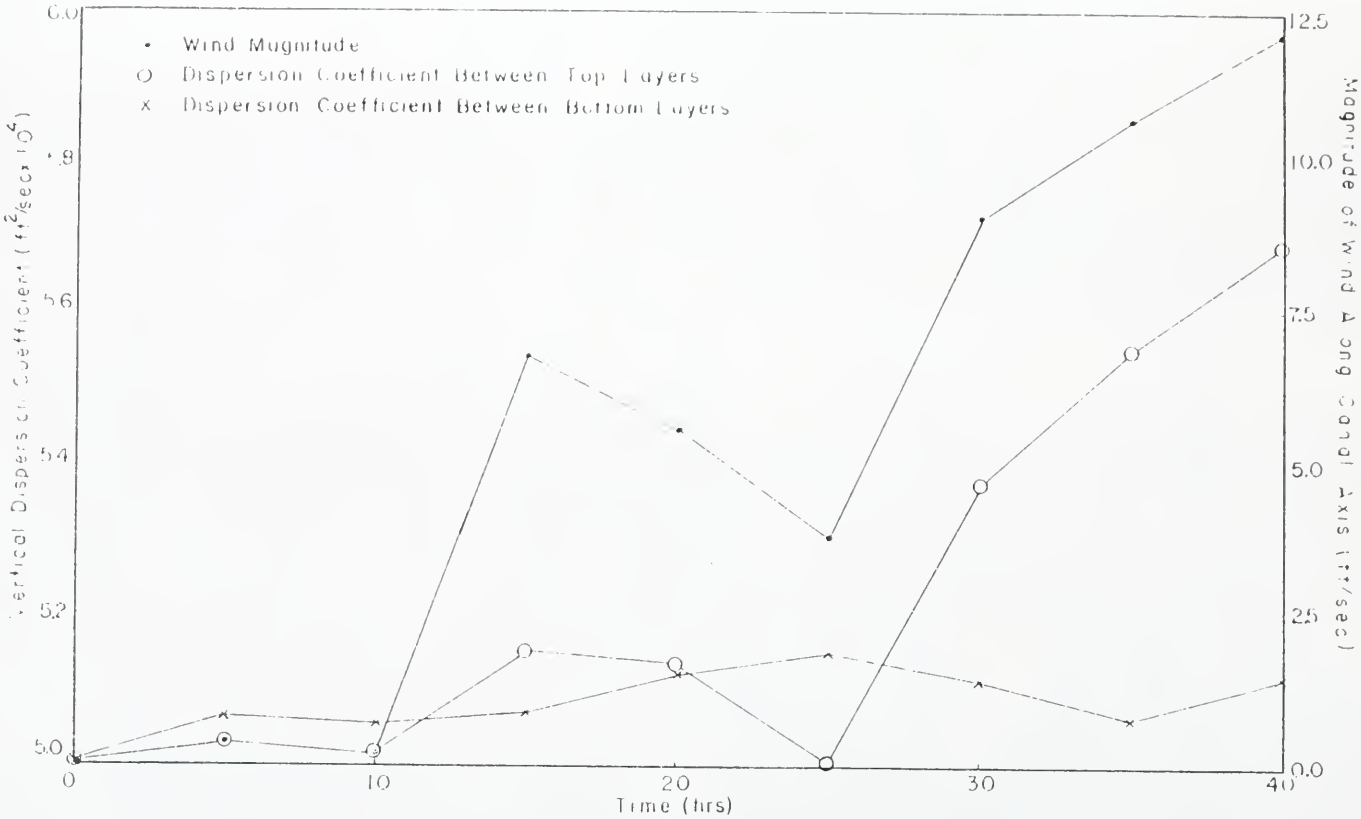


Figure 7.18 - Variation of Vertical Dispersion Coefficient With Time and Wind Speed at Mid-Point of Reach 1 (Figure 7.14), 57 Acres October, 1977 Case.

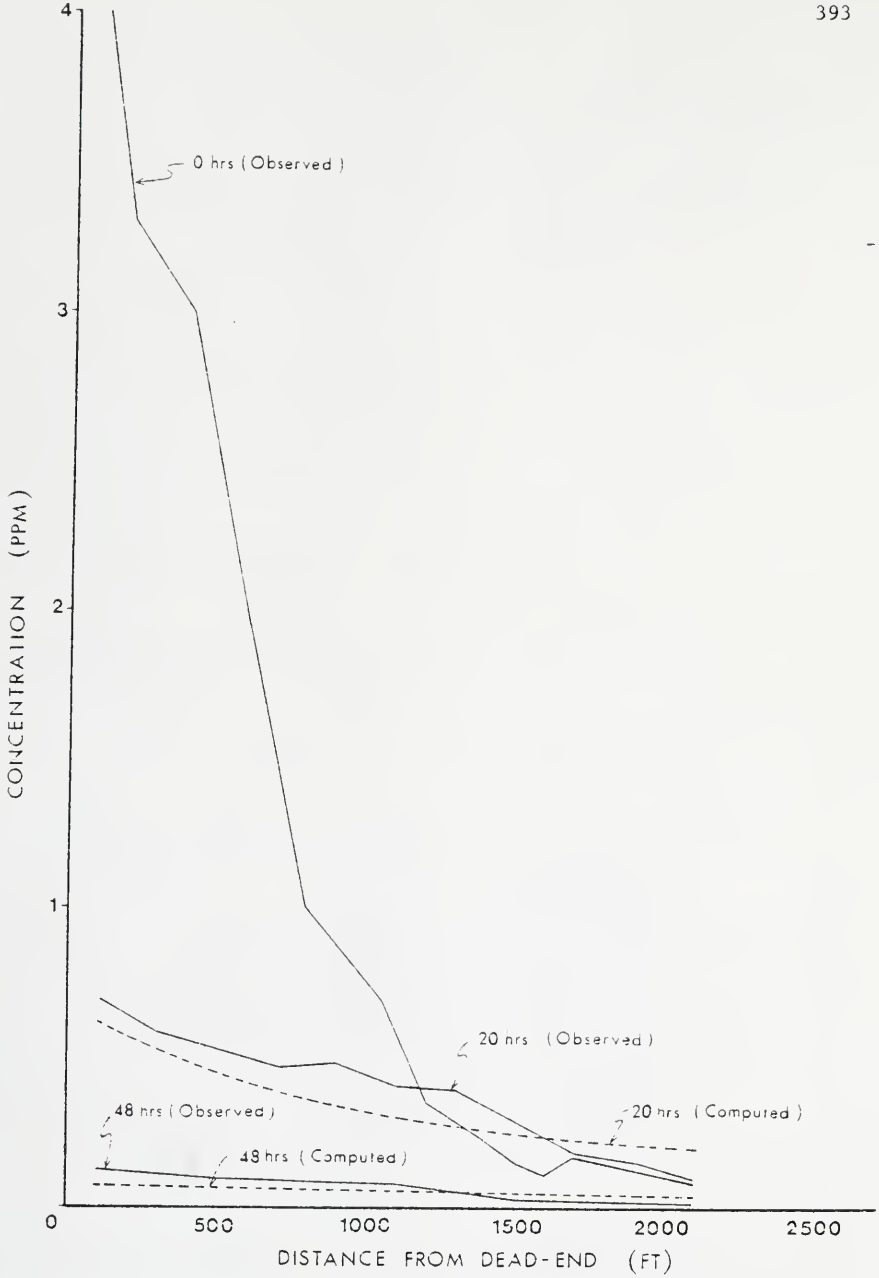


Figure 7.19 - Case No. 3: Comparison Between Observed and Computed Concentration for Loxahatchee River North Canal, June, 1977.

CHAPTER 8

DESIGN OF TRIAL CANAL NETWORK AND VARIABILITY OF DESIGN ELEMENTS

In the first seven chapters the background information and tools needed for designing the hydrodynamic portion of a residential canal system have been described. In this chapter a design philosophy and a procedure for developing a trial canal network will be developed, and the variability of canal network design elements will be described. Chaper 9 will show how the trial canal design can be evaluated and improved using the numerical model, and how the elements in the final network layout are designed to optimize flushing and stability.

8.1 Outline of the Overall Design Process

The overall process of designing a canal network is iterative at the present stage of development. It has been assumed that the principle of superposition applies, which means that the effect of a variation in any one independent variable can be added to the effect of changes in any other independent variable. The assumption is reasonable when considered in context with the assumptions inherent in the derivation of the basic equations for the numerical model, and with the limitations in spatial and temporal resolution in the data that have been used to support the development of this project.

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 prepared a detailed topographic map of the site and the surrounding area. This map should have been annotated with general drainage patterns, the locations of areas of special concern, and the locations of existing canals and waterways. Then, following a set of design guidelines such as those referenced in Chapter 4, 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 8.1.

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 type(s) of materials which compose its bed and banks, its exposure to the wind and its variability (in terms of equilibrium concentration profiles) to changes in geometric parameters. The characteristics of these elements can be generalized and quantified to an extent, and therefore such elements can be assembled in various ways into a canal network to achieve desired flushing characteristics. Once assembled, the elements can then be sized to tune the system so that it meets the objectives established for it. Table 8.1 is a list of major design elements and some of their characteristics. Those characteristics that can be quantified are used later on as design constraints (Section 8.2) and design criteria (Section 8.3).

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, such as the vegetation at the site, should not be changed unless the overall value (both natural and economic) of the site 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 nebulous environmental cost, the short-term cost involved in the construction and marketing of the sites and the long-term expense of the development to the community. In general, it is more expensive (that is, it requires additional energy) to maintain a canal system that cannot maintain itself in relationship to the natural systems with which it interacts.

8.2 Quantifying Design Constraints

Design constraints, as described in Chapter 4, are limitations on the design that derive from external considerations. They include conditions at the site that cannot be changed, requirements imposed by regulatory agencies, and the upper and lower limits of design variables which, if varied, will still satisfy the overall design objectives. For example, there will usually not be a particular depth to which all of the canals must be designed. There may be, however, some data to indicate that the upper strata of the aquifer lie at a depth of, for example, 20 ft below mean sea level. In this case a design constraint would be to permit no canal bottom elevation below - 15 ft MSL. Design constraints are used as the limiting values within which the optimal canal design

will be sought. Some typical design constraints are listed in Table 8.2.

Flushing time may be defined as the time required for an initial, abnormal pollutant point loading (or distribution) to be reduced to some small percentage of the initial value at that point (or at all points) in a canal or a canal network. There is no standard at present for flushing time, since until now the calculation of flushing has been highly subjective and not based on a proven model. The ability of CANNET3D to simulate flushing in canal networks also needs to be verified with more extensive data, although it has been shown to be able to predict measured dye concentrations in both a single, straight dead-end canal and in the South Loop in the 57 Acres canal network. Using this model, a design criterion that required a decrease in the flushing time at one location, or throughout the network, could be established. The amount of such a decrease could be expressed as a design constraint.

The orientation of the axis of otherwise poorly flushing finger-canals, or any other reaches in the network, may be limited to a small sector around the prevailing wind direction by a design constraint. Since the most frequent wind direction shifts seasonally, the orientation of channels will be governed by the season and the location in the network where flushing is expected to be the poorest. The reaches in the network that are expected to have the most difficulty with flushing should be oriented so that the mean wind direction is downstream. However, this may be somewhat difficult to do, in practice, with restrictive topography or limited surface area.

The bank slope of a reach should be designed to minimize the tendency for accretion or erosion according to the grain sizes of bed

and bank materials. Bank slopes of between 1:3 and 1:5 are ideal for planting stabilizing grass and dissipating boat wakes. Flow-through canal stable cross-sections can be calculated using Christensen's method for mild slopes and cohesionless materials [Christensen, 1971; Christensen, 1976]. Specified bank slopes are still another example of design constraints.

The vegative aspects of a canal are one of the most important from a regulatory viewpoint. The applicant for a permit should demonstrate that one or more indigenous species have been selected that have some value to the environment, that these species have a high likelihood of successful establishment, that they will coordinate ecologically with other already-established species, and that they are not types that could overpopulate the area and become a nuisance.

The shorelines of the canals can be vegetated completely except for a relatively small bulkheaded or riprapped portion. To express the structural portions as a design constraint, it may be more convenient to limit these structures on a per-lot basis (for example, "no less than 70 percent of the waterfront of any lot should be vegetated slopes") than to express some overall percent limit for the total length of shoreline of the network.

Vegetated shallows are usually associated with a large waterbody such as a lagoon or basin. Access over shallows for boating purposes can be accomplished with docks built on pilings, so unless there is a potential erosion problem all of the shallow areas could be planned to be vegetated. The design criterion for vegetated shallows may be expressed in terms of a percentage of the total shoreline of the network, and the width of the sloped side to be planted.

8.3 Quantifying Design Criteria

Design criteria, as described in Chapter 4, are statements specifying the value (or range) of a variable that is used in a canal design. The canal design can be considered to be made up of design *elements*, such as straight reaches, bends, junctions and branches, all of which must be dimensionalized through the various design criteria. Table 8.3 lists some examples of design criteria.

Beginning with overall system design criteria, a design storm is specified that meets local and state criteria, if any apply. If a portion of the property lies in a flood plain, it will be advisable to design for at least the 25-yr flood plain. Outside the flood plain storm water retention could, for example, be designed for the 100-yr storm, and drainage without retention for the 25-yr storm.

The location and orientation of one or more tidal entrances will be dictated by the location of existing channels and entrances (if any), by the topography of the site, and by the bathymetry of the communicating tidal waters. This can usually be decided in the early stages of design. Tidal entrances in general should have gradually changing surface and cross-sectional areas to minimize loses. Intertidal and supratidal vegetation can be specified from known conditions at the site.

Lagoons or basins are an important design element, since they can be used to adjust the desired discharge in the system. They can generally be located during the initial layout, and approximately sized. Islands inside lagoons not only provide a method for conveniently tuning the network, but also are a valuable aesthetic feature.

Sinuous, or meandering, canals have better mixing characteristics than straight canals and offer the additional advantages of greater

privacy for each lot, since if lots are properly staggered on the curves the view from any one lot bulkhead to neighboring bulkheads can be obstructed (see Figure 8.2, a concept originally conceived by R. Snyder for the 57 Acres development). Another design criterion that can be established is a requirement that all spatial transitions must be gradual, and that no sills are permitted. Initial values for channel geometry should also be established at this stage, although they may have to be changed as the design evolves.

8.4 Trial Canal Design

The objective of a trial canal design is to develop the canal network layout, channel geometry, and specific values for the independent variables required for operating the mass-transport canal model. Two general classes of problems can be identified at the outset, the first being the case in which the site has no existing canal network, and the second in which there is an existing network. At the time the design of the trial canal is initiated, it may be assumed that in the first case sufficient measurements have been made to determine where circulation or flushing problems exist in the network. Then, the problem is to vary the existing design until an acceptable channel geometry, and if necessary an acceptable new network design, has been obtained. The iteration of the design toward an optimal form is described in Chapter 9.

8.4.1 Topographic Site Map

Before network layout can proceed, it will be assumed that a topographic map with natural drainage patterns, existing waterways, reserved areas, natural areas of soils and vegetation to be preserved,

and existing roadways, utilities, dwellings, boundaries and other special features of the site has been prepared. The scale of such a map should be 1:2400 (more or less, depending on the overall size of the site). This map could be taken from an aerial photograph, and/or by surveying, and should overlap the site by whatever extra area is required to permit significant external influences to be assessed. An SCS soils map should also have been obtained. A scale of 1:20,000 is typical for the SCS maps and should be adequate for generating a synthetic storm hydrograph. In general, however, a soils map to this scale is not adequate for making decisions relative to canal layout and drainage.

8.4.2 Storms, Hydrographs and Pollutographs

A time-varying freshwater flow, with or without a time-varying concentration, can be input into any cell in the two- or three-dimensional mass-transport model (CANNET3D). This inflow could represent tributary flow, flows of pollutants, or in general the input or outflow of any constituent which can be described by a first-order decay coefficient. Unless local or regional legislation requires otherwise, the 25-yr design storm should be used for the residential canal design. The design storm is defined by rainfall duration (minutes) and corresponding intensity (inches/hours) for the given return period. These relationships are published for 1 to 100-yr return periods and durations of 30 min, 1 hr, 2 hr, 3 hr, 6 hr, 12 hr, and 24 hr for any location in the state [Soil Conservation Service, 1973].

The drainage basin associated with the site should be located on the appropriate 7-1/2 min USGS quadrangle map. The water divide consists of the locations of the highest points that control the flow into or out of the area contiguous with the site. Often roads, railroads,

ditches and some natural features will be found to partially define the water divide. The drainage basin for the site is, of course, limited to the area that produces a definable freshwater inflow to the site or the waters bordering the site to an extent that would affect either the drainage or the functioning of the canal network. The synthesis of hydrographs is described in detail in the literature [Hjelmfelt and Cassidy, 1975; Viesman et al, 1972; SCS, 1972]. An alternative numerical approach by Christensen, et al [1974], was developed for defining the 100-yr flood plain for a metropolitan area. It is based on a numerical model which, from a definition of subbasins, develops a storm hydrograph for each subbasin and routes the flood through channel networks, detention and retention basins, and control structures. Infiltration and overland flow are accommodated by determining the areas associated with each type of ground cover in each subbasin, using the rational method. This model takes some time to set up on a computer, as the drainage basin characteristics must be defined in detail. However, if drainage patterns are complex, or detention basins and control structures have a significant influence on the problem, this method provides a procedure for obtaining a more realistic hydrograph than the SCS method.

A "pollutograph" is similar to a hydrograph but shows pollutant concentration, c, instead of discharge, Q, as a function of time. No specific method for generating such an input has been developed for this project, but urban storm water models such as EPA's SWMM [1975a] would be suitable.

8.5 Variability of Parameters in a Straight Reach

It has been shown that the hydrodynamics in a finger canal network under the influence of the tide and wind, salinity gradients, secondary currents, and/or lateral inflows are three-dimensional. However, a study of the variability of geometric parameters in a straight, dead-end canal under the influence of tide alone can be useful to the design engineer because the effects are fundamental and it can be assumed that they are still valid in the presence of the additional forcing functions.

The variability of the equilibrium concentration profile in a trapezoidal, straight, dead-end tidal canal was studied using a one-dimensional mass-transport model with negligible numerical dispersion. All the variability simulations are based on the test canal network shown in Figure 8.3, with values from the set summarized in Table 8.4. The varied parameters may be divided into noncontrollable and controllable parameters. The first category includes the tidal amplitude, a, and the entrance decay coefficient, τ, associated with the flood tide tidal entrance boundary condition. Figure 8.4 shows that the decay coefficient has a surprisingly small effect on the equilibrium concentration profile in the canal up to a period corresponding to the half tidal period. This result shows that any inaccuracies in the selection of the value of τ for the flood tide boundary condition do not have a significant effect on mass transport in the canal. On the other hand, variations in the tidal amplitude, a (Figure 8.5), show a much greater effect, as would be expected from consideration of the effect of changes in the tidal prism. It can be seen that uniform increases in the tidal amplitude have a decreasingly greater effect on the equilibrium concentration profile.

The second category of parameters, the controllable ones, include the geometric variables length, width, mean tidal depth, and inverse side slope, as well as the rate of lateral inflow and its concentration. Nikuradse's equivalent sand roughness and the dimensionless longitudinal dispersion coefficient were also varied over their natural range in Floridian tidal canals.

Changes in the geometric parameters, with the exception of the mean tidal depth, give results that seem to be intuitively obvious. The effect of increasing the length, L, of the canal is a linear increase in the concentration closer to the dead-end (Figures 8.6 and 8.7) since the tide has a limited excursion distance and its flushing effect becomes smaller with greater distances from the entrance. An increase in the bottom width, b (Figure 8.8), or the inverse side slope, s (Figure 8.9), directly decreases the equilibrium concentration profile due to the increase in the volume of water introduced from the receiving waterbody for dilution of the pollutant inflow. Conversely, the effect of an increase in the rate of lateral inflow (Figure 8.10), or the concentration of the inflow (Figure 8.11), is a proportionate increase in the resulting equilibrium concentration profile due to the mixture of a greater volume of pollutant with the same volume of water.

The effect of changes in the mean tidal depth is shown in Figure 8.12. An increase in the depth appears to decrease the equilibrium concentration profile, again due to the increase in the tidal prism. However, this result is somewhat misleading, as other important considerations have not been included in the analysis. Since an increase in depth increases the distance over which dissolved oxygen must diffuse to reach the bed, and decreases the amount of light that reaches the bed,

the deeper canals are more likely to tend toward anoxic conditions in the absence of the other forcing functions previously listed. Thus, the results of this variability analysis should not be used in the evaluation of a canal network, but rather the more realistic two-dimensional model results with wind effect should be used.

The effects of increases in Nikuradse's equivalent sand roughness, k (Figure 8.13), and the dimensionless dispersion coefficient, K (Figure 8.14), are as would be expected. Equation (7.10) for the longitudinal dispersion coefficient shows that increases in both parameters increase the longitudinal dispersion coefficient, the roughness having somewhat less influence due to its presence in the natural logarithm term.

Tests of variability were also conducted with a second model canal (Figure 8.15) consisting of a main 1000 ft canal and a 500 ft branch located at various positions along its length. The resulting concentration profiles at low and at high tide, for location of the branch canal at distances, x, of 200, 400, 500, 600, and 800 ft from the dead-end are shown in Figures 8.16 and 8.17 respectively. The results indicate that as the branch canal is located progressively closer to the dead-end, the resulting equilibrium concentration profile in the main channel is reduced as the effective excursion distance of the tidal prism is increased. However, the opposite appears to occur for the branch canal and design decisions should be made on other factors.

8.6 Variability of Parameters in Three-Dimensional Mass-Transport

The previous section has summarized the effects of different parameters in the one-dimensional mass-transport model. Most of the parameters varied in those tests were related to the geometry of the canal, and thus their qualitative effect will be the same for the

three-dimensional model. This section deals specifically with the effect of wind-induced circulation and density currents on a simple system. The analysis is based on an arbitrary canal example somewhat similar to the Big Pine Key canal III (Section 7.5.1). The longitudinal axis of the canal is aligned to the south (180 degrees). The tests were conducted by Walton [Morris, Walton and Christensen, 1978, pp. 353-361] using the two-dimensional version of CANNET3D, and the test parameters are summarized in Table 8.5. Only the results after 50 tidal cycles are shown here, although the effectiveness of each wind condition over time will be described.

8.6.1 Effect of Wind-Induced Circulation

The effect of wind-induced circulation in the absence of density currents will be covered in this section. For each of five cases, 1W to 5W, five different wind conditions were used:

Wind Speed and Direction
no wind (0, 0)
5 mph downstream (5, 0)
5 mph upstream (5, 180)
10 mph downstream (10, 0)
10 mph upstream (10, 180)

In the above table and the figures showing the results of these tests, the wind speed and directions are labeled "(wind speed, direction)."

For the first test (1W) the initial concentrations in the canal, c_i, were uniformly set at 100 ppm and the background concentration, c_{RW}, at 5 ppm. The resulting vertically-integrated concentration profiles after 50 tidal cycles are given in Figure 8.18. As one would expect, the concentration profile for no-wind shows very little flushing. Complete flushing would take many hundreds of tidal cycles under no-wind conditions, as has been shown in addition by all of the one-dimensional

mass-transport models tested for this project. The 5 mph downstream wind has flushed over half of the substance out of the canal after 50 tidal cycles, whereas the upstream 5 mph wind is not quite (about 90 percent) as effective. This is due to the lower transport rate in the bottom layer, caused by friction at the bed and the upstream movement of the substance. The 10 mph wind was able to flush the substance almost completely in fifty tidal cycles.

In the second test (2W) the two concentration values were interchanged, the initial concentrations becoming 5 ppm and the inflow concentration being set at 100 ppm. The results after 50 tidal cycles (Figure 8.19) are opposite to those of case 1W. For zero wind hardly any intrusion is found, while for a wind speed of 10 mph the concentration in the canal almost reaches the background value after 50 tidal cycles. For a 5 mph wind, the upstream wind direction is not as effective as the downstream wind direction.

In the last three cases a lateral inflow with concentration, c_I, of 100 ppm is simulated and the flow rate is varied from one case to the next while the total volume influx is held constant. In case 3W the uniform lateral inflow is 0.04 $ft^3/hr/ft$, and the resulting concentration profiles after 50 tidal cycles are shown in Figure 8.20. Under no-wind conditions the concentrations at the dead-end are steadily increasing (as determined by comparing results after 5 and 50 tidal cycles), and a wind blowing out of the canal is much more effective in flushing than an upstream wind. This results from the form of the wind-induced vertical velocity profile, since the upstream wind transports the substance towards the dead-end in the upper layer, and then down and toward the entrance in the two lower layers. Thus the substance has farther to go

to reach the tidal entrance and takes much longer to do so than for the corresponding 5 or 10 mph downstream wind.

For case 4W the lateral inflow rate has been doubled over the inner-half of the canal only, so that the total mass inflow is the same as for case 3W. The results after 50 tidal cycles are shown in Figure 8.21. For no wind, or a wind blowing into the canal, the increase in concentration at the dead-end is higher than in case 3W. A 10 mph downstream wind is still able to achieve some degree of flushing over time and the increase in concentration along the canal in this case is relatively small. The 5 mph wind blowing out of the canal is not as efficient as in the previous case (3W) because more of the substance is diffused down into the lower layers, due to the greater concentration gradient near the dead-end, and transported back towards the dead-end.

In the final case, 5W, the lateral inflow has been increased four times but limited to the 200 ft section at the dead-end. The resulting concentration profiles after 50 tidal cycles are shown in Figure 8.22. In each case the wind-induced circulation was unable to reduce the continuous loading over a period of time, although the downstream winds were more effective in flushing than in the preceding cases. In this case, however, the 5 mph downstream wind was not as effective in flushing as the 10 mph upstream wind. This result indicates that there is some set of conditions in the variability analysis at which the transfer of substance vertically downward, due to an upstream wind, becomes more important in flushing than pure convective transport downstream. In this case, the 5 mph wind blowing out of the canal is not able to transport the substance quickly enough to prevent considerable vertical transfer to the lower layers and subsequent transport toward the dead-end.

In conclusion, it is noted that most cases of lateral inflow into canals occur into the surface layer, such as run-off from lawns and leaching from septic tank drain fields. In these situations, a downstream wind is more effective in flushing than an upstream wind. Also, in cases where the concentration in the receiving waterbody is higher than the desired concentration in the canal, a downstream wind is more effective in flushing the canal than an upstream wind. However, if the main source of pollutant is in the lower layers, and sufficiently far from the dead-end that pollutants will not accumulate there, then a wind blowing into the canal might be desirable over a downstream wind to obtain optimum flushing.

8.6.2 Effect of Density- and Wind-Induced Circulation

In this section, a density-induced circulation superimposed on a wind-induced circulation is considered for 5 test cases, 1S through 5S. The conditions for the tests were the same as for the correspondingly numbered test 1W through 5W, as shown in Table 8.5. For these comparisons only two wind conditions will be used, 5 mph upstream and 5 mph downstream, with and without superimposed salinity variations.

The salt wedge interface in these test simulations was located 5 ft below the mean tidal elevation at the tidal entrance, and the coefficient u_4 in Equation 7.43 was set at 4000. The results are plotted only after 50 tidal cycles, as in the previous section.

The results for case 1S are shown in Figure 8.23. In comparison with case 1W, the movement of the salt wedge in and out of the canal induces greater circulation and produces lower concentration profiles. As in case 1W, a downstream wind is more effective in flushing than an upstream wind. A wind blowing out of the canal induces an upstream

current in the lower layers which adds to the velocities associated with the movement of the wedge, on flood tide, which in turn induces greater vertical mixing across the interface. Conversely, on the ebb tide the lower-layer velocities tend to cancel. The net effect, however, over many tidal cycles is substantially greater flushing in the presence of the density-induced flow.

In case 2.S (Figure 8.24), in which the concentration of the receiving waterbody is higher than in the canal, the situation is reversed. Since the movement of the salt wedge dominates the velocity field at lower values of wind speed (around 5 mph), much more of the higher-concentration water from the receiving waterbody is circulated into the canal. In this case a wind directed into the canal is preferred, since the reversal in the lower layers aids in flushing.

In cases 3S to 5S results (Figures 8.25 through 8.27) show that the density currents have a progressively smaller influence on the concentration near the dead-end, but they do improve flushing to some degree (except in case 3S, when the 5 mph wind is blowing out of the canal). The wedge-induced circulation increases the flow out of the canal in the upper layer, and also increases the vertical dispersion into the lower layers.

In conclusion, it appears that in general the presence of a salt-water wedge in a canal will improve flushing to some degree. However, the results also show that if the concentration of a substance is higher in the receiving water than in the canal, flushing will be less effective than with wind-induced circulation alone.

The results in this and the previous subsection provide only a qualitative view of the effects of wind and a salt wedge on the flushing

characteristics of a canal. Since the velocity field depends on the astronomical tide, the magnitude of the wind, and the height and extent of the salt wedge, mass-transport and mixing in the canal can vary greatly with different conditions and from flood to ebb, so that a comprehensive analysis is not feasible.

8.7 Variability of Canal Network Design Elements

In this section several simple canal networks will be described and compared on the basis of simulation results. These networks may be considered as design elements, each having certain characteristics that can be expected to apply when a larger system is constructed from various combinations of these elements. The geometric variables and coefficients are constant from one system to the next, so that comparisons can be made between the results in a variety of ways. Figure 8.28 shows the arrangement of these network designs, and their dimensions are summarized in Table 8.6.

The dimensions of the reaches were obtained by measurement of typical canals in a residential canal development. Each dead-end reach is 2000 ft long, which will accomodate thirty-four rectangular lots with 100 ft frontage and 200 ft depth to the street, six irregular lots at the dead-end, and three lots at the mouth (Figure 8.29). Assuming a 50 ft wide street right-of-way between canals and 100 ft wide canals, the centerline separation between canals is 550 ft.

The trapezoidal section used for these systems is shown in Figure 8.30. The 50 ft bottom width and 8 ft mean depth are typical for Floridian canals. With a side slope of 3:1 the mid-tide surface width is nearly 100 ft.

8.7.1 Design Tests

The principal variables used in the design tests in this chapter are wind and pollutant inflow. A wind-component blowing *into* a dead-end reach causes the surface layer to move toward the dead-end, where pollutant is carried downward into the middle and bottom layers and is transported by the velocities in those layers. The effect of a downstream wind component is to carry pollutants out of the canal at a substantially greater rate than occurs with only tidal convection, and faster than for an upstream wind component. The quantitative differences in wind effects in a straight reach have been shown in Sections 8.6.1 and 8.6.2. Since the concentration of the no-wind distribution decreases as a function of the wind speed, it follows that the channel with the greatest potential for pollutant build-up should be aligned as closely as possible with the prevailing wind, with the tidal entrance downwind of the prevailing wind direction.

The simulations of these canal networks begin with a zero or one ppm background concentration, depending on the simulation to be conducted, equal to the receiving waterbody concentration at all points in the network. The pollutant inflow was set at a discharge of 0.04 ft^3/ft-hr at a concentration of 100 ppm at each section in the network. The high tide concentration above background at the midpoint of the first section in each reach is plotted and labeled either as a "junction" value or as a "dead-end" value. The basic objectives of these simulations are to observe and explain the flow mechanisms in the networks, to compare surface and bottom concentrations at certain points in the networks and along a representative finger canal, and to relate these comparisons to wind conditions.

8.7.2 Simple Comb-Structured Canal (System A)

System A consists of a series of four parallel, straight reaches joining a straight channel at ninety degrees. The "south" end of the main channel is a tidal entrance, and the network can be considered to be a comb-structured canal system (Group 4, Section 2.4.1). The dimensions of the channels and the designation of reaches and junctions are shown in Figure 8.31. For convenience in discussing relative wind directions, all networks in this chapter are considered to be oriented with the tidal entrance toward the south.

Under no-wind conditions System A has not reached an equilibrium after thirty tidal cycles. However, several characteristics are apparent which would not change with increased simulation time. Figure 8.32 shows surface and bottom concentrations at the four interior junctions and at the dead-end of each finger canal at high tide. The surface concentrations are substantially higher than the bottom concentrations at the two junctions farthest from the tidal entrances. This results from the introduction of the pollutant into the surface layer only, and the amount of vertical mixing is essentially controlled by the vertical diffusion and momentum transfer coefficients (which are set at the values found to best fit the Loxahatchee north canal and 57 Acres canal network). As expected, the increased tidal velocities closer to the entrance induce greater vertical mixing and there is relatively little difference in surface and bottom concentration values. Also, junctions number 4 and 5 appear to have reached their equilibrium concentration values within thirty tidal cycles.

The concentration values at the dead-ends of the finger canals are all the same at the surface, as they are in the bottom layer. This

result is to be expected since, with the horizontal water surface assumption, the velocities are determined solely by the change in upstream tidal volume.

The superposition of a 5 mph wind blowing directly into the finger canals (a west wind, in this case) gives the results summarized in Figure 8.33. The simulations have been continued for fifty tidal cycles, and no other changes have been made to the network from the no-wind case. The decrease in concentration at the dead-ends of finger canals progressively closer to the tidal entrance occurs because the ebb tide is able to flush more pollutant mass out to the receiving water, and the flood tide excursion intrudes farther into the closer canals.

The surface concentrations near the junctions resulting from the upstream wind are slightly, but not significantly, lower after thirty tidal cycles than for the no-wind case. The surface and bottom values at the dead-ends are substantially lower for this wind condition (approximately 40 percent at reach number 1 dead-end), reflecting the flushing effectiveness of even an upstream wind. Figures 8.34 shows the differences in concentration profiles along this reach for the no-wind and wind conditions. Under the influence of a wind blowing toward the dead-ends the surface layer concentrations are convected vertically downward at the dead-end and into the lower layers, and in this case they are convected out of the canal in the lower layer more effectively than in the no-wind case (except near the mouth of the canal).

The results of changing the wind direction to the downstream direction are shown in Figure 8.35. In this case the concentrations at all reaches and junctions are lower than in either of the preceding cases, as would be expected. The decreasing concentrations with reaches closer

to the tidal entrance show that pollutant mass is being flushed out of the system at a faster rate. Figure 8.34 shows this effect quantitatively in reach number 1.

Figure 8.36 is a comparison of the concentrations at the dead-end of reach number 1 and at junction 2 as a function of time for both surface and bottom layers. It is evident from these curves that the system in each case is progressing toward an equilibrium condition, but the time rate of change for these conditions is so slow that no additional information will be gained by extending the simulation.

8.7.3 Comb-Structured Canal With Lake

System B is the same as System A except for the addition of a large body of water at the dead-end of the "northern" reach. The surface area of the "lake" 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 simulations to follow this is unnecessarily large, resulting in more than adequate flushing of all parts of the network except the inner halves of the finger canals. The lake acts as a source and sink of water for the network, so that a variable tidal prism can be used to tune the flushing of the network, but does not include any circulation or variation of concentration within its boundaries. The layout and dimensions of System B are shown in Figure 8.37.

The tidal volume of the lake increases the tidal prism of the network, which effectively flushes the north reach (number 1) and the main channel to the tidal entrance (Figure 8.38). A direct comparison between this case and no-wind conditions for System A (Figure 8.32), considering that the latter was only simulated for thirty tidal cycles, shows that there is effectively no difference in concentration profiles

at the dead-ends of the finger canals, again because the tidal velocity is a function of only the upstream tidal prism and distance from the dead-end. However, the concentration profiles for reach number 3 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. This can be seen by comparing Figures 8.39 and 8.34.

The effect of superimposing a wind blowing into the finger canals is summarized in Figure 8.40. Again, as seen for System A, a downward circulation at the dead-end carries pollutant mass into 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. Figure 8.39, showing the concentration profiles in reach number 3, demonstrates that the concentrations are uniformly small throughout the reach for an upstream wind.

A downstream wind is even more effective in flushing pollutants from this system. Figure 8.41 shows approximately the same pollutant concentrations at the surfaces of junctions, but an order of 80 percent lower concentrations in the bottom layer and an order of 50 percent lower concentrations 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 3 shows this to be true in the surface layer in the inner half of the reach, and throughout the bottom layer.

8.7.4 Comb-structured Network With Bends

The third network, System C, (Figure 8.4.2), is a second variation on the simple comb-structured network (System A). This network is

identical to System A except that the four parallel finger canals are curved. The length along each curve is the same as the length of the fingers in System A (Figure 8.31).

The curved reaches are specified by the radius of curvature, r, and the centerline length of the bend, L. It is convenient to find the radius of curvature by specifying an offset distance, D_o, of the crown of the bend from a straight line connecting the ends of the curved reach. Then the angle, α, included between the radii to the ends of the curved reach is found by solving iteratively the expression

$$\frac{\alpha}{1 - \cos\frac{\alpha}{2}} = \frac{L}{D_o} \tag{8.1}$$

where

α = included angle between radii, [rad]

L = length of bend along centerline, [L]

D_o = offset distance, [L]

and the radius is given by,

$$r = L/\alpha \tag{8.2}$$

In the example described in this section, an offset distance of 200 ft resulted in an angle, α, of 0.811 rad and a radius of 2466.1 ft.

One simulation was made with System C for a wind blowing directly upstream along the chord of the finger canals. Figure 8.43 shows the results of this simulation, in which it is apparent that the concentrations (averaged over the three cells in each layer) are somewhat higher than those in the straight finger canals, Figure 8.35, in both the surface and dead-ends. The explanation for these results stems from the lateral structure of the three-dimensional model.

In the three-dimensional model pollutant can be introduced into the two sets of surface cells along the sides of the channel. Since a trapezoidal channel cross-section is simulated in this case, the velocities in the vertical side planes are less than the corresponding velocities in the centerline plane. The pollutant inflow is not instantly averaged across the channel as it is in the two-dimensional model, but must instead be convected by the helical flow in the bend and diffused according to the magnitude of the lateral diffusion coefficient. As a result, the concentrations tend to be somewhat higher along the sides, and are convected and mixed more slowly throughout the channel cross-section, than in the two-dimensional case. Since the three-dimensional version of the model is more realistic in its representation of the details of the flow through the network, it is expected that its mixing characteristics are likely to more realistic as well. However, if a curved reach is used anywhere in the CANNET3D model, the geometry of the entire network must be set up in three dimensions. Since this triples the cost of a simulation, it is important to carefully evaluate whether the additional information obtained from a three-dimensional simulation will have a commensurate effect on resulting design decisions. This is ultimately a value judgement to be made on the basis of the extent and quality of field data obtained at the site, and the presence of any flushing characteristics in the network which cannot be satisfactorily analyzed in two dimensions.

8.7.5 Simple Network With Two Tidal Entrances

If a canal system has more than one tidal entrance, and the entrances are spaced sufficiently far apart to result in a significant head difference between them, the flow in the network can no longer be

accurately described by the horizontal water surface assumption. A numerical model based on the momentum equation would be required in this situation. However, if accurate measurements of tidal heights and corresponding times are available for the entrances, calculations of average flows and velocities using the tidal prism approach will permit some design estimates to be made (for example, see Morris, Walton and Christensen, [1975]).

A special case of this situation, however, can be accommodated with the CANNET3D model if the tidal heights are in phase at the entrances or are close enough in phase that the phase lag can be accommodated by shortening the length of the entrance reach in which the leading tidal component is found up to, but not farther than, the first junction. This reach should be shortened by an amount

$$L = \sqrt{gd_o}\ t \tag{8.3}$$

where

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

d_o = mean depth, (L)

t = time associated with phase lag, (T).

which then permits the use of the same tidal information at each entrance. The null point would then be located at that point which divided the network into two equal surface areas.

System D, shown in Figure 8.44, is a symmetric comb-structured network which is constructed by connecting two type "A" systems through the upper or "northern" reach. This reach has been extended an additional 400 ft to provide a realistic separation between the dead-ends of the other six finger canals, but otherwise the dimensions are the same

as for System A. The null point is assumed to be located at the midpoint of the connecting reach, at the point where reaches number 8 and 9 join. The model is designed to have a zero tidal velocity component at this point, but to simulate any wind-induced mass-transfer through this area.

The results of no-wind conditions on this network are shown in Figures 8.45a and 8.45b. Since only tidal flow is considered in this case, the concentrations should be practically the same at all junctions and dead-ends as at the corresponding points in System A (Figure 8.32). The comparison in Table 8.7 for both no-wind and east wind conditions shows that the concentration values at the junctions are slightly lower in System D than in System A, and the concentrations at the dead-ends are virtually the same in both networks. These differences can be attributed in part to the slight increase in distance to the null point in the northern reach, and to numerical errors inherent in computations with extremely small numbers.

The results of the simulation with an east wind are shown in Figures 8.46a and 8.46b. In this case, both surface and bottom concentrations are on the order of half of the values in the no-wind case.

8.7.6 Summary of Observations on Network Design Elements

The relative flushing characteristics of simple canal networks can be predicted, on a qualitative basis, on the basis of the comparisons described in this chapter. The tidal excursion distance, x', which is approximately given in terms of the total length of a canal, L, by Equation (2.3) provides an *estimate* of the distance to which tidal flushing will be effective in a network with dead-ends and no "lakes." Thus, 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 then 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 downstream in the bottom layer occurs under these conditions, but flushing is slow. The flushing of the main (in this case, the north/south) canal is entirely by tidal action in these simulations since there is no north/south wind component.

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 (at least within the tidal excursion length) and upward flow at the dead-end will provide some vertical mixing.

The addition of even a small "lake" 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 downstream portions of finger canals, due to increased convection, but little effect on dead-ends (except, of course, the dead-end at which the lake is connected).

Reaches with bends provide better vertical mixing, as shown in previous chapters. This effect can be extrapolated to finger canals in networks. It was shown that the concentrations at junctions are only slightly increased in the case with bending finger canals, which may be attributed to differences between the two- and three-dimensional models.

Symmetric networks with two tidal entrances can be analyzed provided that a null point can be located. For no-wind conditions the two halves of the network have the same characteristics, and are independent. With a wind, the concentrations in System D are approximately the same as for System A, considering computer limitations in manipulating small numbers.

Table 8.1 - Elements of Design.

Element	Characteristic	Feature	Effects, Advantages and Disadvantages, and Design Constraints and Criteria
Tidal Entrance	Orientation	with canal	Orientation in direction of channel minimizes bend-induced losses. Orientation at an angle will induce stronger helical flow (see "bends").
		with receiving waterbody	Orientation affects visibility between boats entering and exiting.
	Cross-section	Flow and stability	Width and depth determined by mean tidal flow, bed slope, side slope, bottom roughness, and effective grain size and critical time-mean horizontal shear stress. Smaller cross-section leads to greater velocity and higher probability of scour.
		Spatial transitions	Gradual spatial transition in area (as opposed to sudden changes) minimizes energy loss and deposition by secondary eddies.
	Multiple entrances		Multiple entrances to system improve flushing if difference in tidal height is on the order of tenths of feet, and/or difference in phase is on the order of ten minutes. If there is no tidal difference at the mouth, the flow, velocity, and flushing in each main channel are reduced by an amount proportional to the number of entrances.
Canal Reach	Width	Tidal Prism	Tidal prism increases proportionally with width of canal. Width is therefore a principal flow design parameter.
		Flushing	Rectangular channel cross-section: Flushing increases logarithmically with width, with significant increases to about 90 ft width (result of variability tests with hybrid model).
		Land area	Increasing width decreases lot size or number of lots.
		Exposure	High trees along banks tend to inhibit wind-induced flushing.

Table 8.1 - continued.

Curves and Bends (continued)	Helical currents (continued)	Design criteria	Curvature should be great enough to induce helical flow, small enough to avoid separation. Energy used to maintain helical flow is proportional velocity.
	Sharp Bends		Additional downstream energy losses due to flow separation. Repeated increases and decreases in velocity caused by multiple sharp bends results in high energy losses.
Dead-end Reach		Flushing	Low velocity inhibits tidal flushing. Wind aligned toward entrance maximizes surface flushing, and induces upward circulation at dead-end. Bottom layer is carried toward dead-end. Wind aligned toward dead-end causes accumulation of surface trash and downward circulation at dead-end. Contaminants in surface waters are mixed into lower layers. Flushing is generally poor.
		Exposure	High trees along canal banks tend to inhibit wind-induced flushing.
Lagoon or Basin	Tidal Prism		Tidal prism is increased by volume of lagoon or basin. If located far from entrance, lagoon may not flush well unless its entrance is designed to maintain clockwise flow. Lagoon or basin should preferably be located between two reaches so that flow is through it, rather than the basin acting as a dead-end.
	Intertidal shallows	Multiple uses	Increase tidal prism. Increase productivity of canal system. Encourage diversity of aquatic life. Islands provide increased lenght of shoreline, flow control, interesting view.
Expanding or Contracting Section		Energy losses	Depending on abruptness of section change: Expansions: energy loss varies from 20 percent to 50 percent of velocity head difference. Contractions: energy loss varies from 10 percent to 50 percent of velocity head difference.

Table 8.1 - continued.

Obstructions	Bridges		Increase local mixing and dispersion. Increase local scour. Dissipate some kinetic energy in the flow. Can limit navigation. Attachment surface for sessile organisms.
	Dock	Mixing	Increase local turbulence, mixing and friction losses.
	Pilings	Scour	Increase local scour.
		Habitat	Attachment surface for sessile organisms.
Development chronology		Sedimentation	Limits sedimentation during dredging operations. Provides protection of land surface during construction.

Table 8.2 - Typical Canal Design Constraints.

Design Element	Typical Design Constraints
System Flushing	Flushing of Normal Pollutant Load (flow, Q_I, and concentration, c_I, distributions specified) to be less than (specified) number of hours under normal climatic conditions.
Tidal Entrance	Orient axis $\pm$ 10° with respect to prevailing wind if possible.
Lagoons and Basins	Maximum size specified.
Channel Geometry	Orient axis of low-flushing channel $\pm$ 10° with prevailing wind.
	Surface width 100 ft or less due to lot requirements.
	Depth between 4 and 10 ft.
Vegetated Shallows	No less than 20 percent of total network shoreline shall be vegetated shallows.

Table 8.3 - Examples of Typical Canal Design Criteria.

Design Element	Typical Design Criteria
Drainage and retention	System is to handle drainage from the 25-yr, 6-hr storm. System is to provide retention capacity for the 100-yr, 6-hr storm.
Tidal Entrances	One tidal entrance at specified location. Orientation specified. Entrance gradual with intertidal vegetation.
Lagoons	Two lagoons at specified locations. Island inside one lagoon. Maximum size specified.
Canals	Sinuous. No bends greater than 30°. Sloping vegetated banks except 30 ft bulkhead at each lot. Lots to be staggered. No sills. Zones of different depths as indicated, with gradual transitions. Depth at MLW : 6 ft Side Slope : 1:5 — Trial values Bottom Width : 40 ft — to be adjusted Surface Width: 100 ft — for stability

Table 8.4 - Standard Data Set for First Test Canal.

PARAMETER	VALUE
Length, L	1000 ft
Bottom width, b	90 ft
Mean tidal depth, d_o	10 ft
Inverse side slope, s	0 ft
Tidal range, a	2 ft
Tidal period, T	12.42 hrs
Nikuradse's equivalent sand roughness, k	5 ft
Dimensionless dispersion coefficient, K	10
Lateral inflow, q_I	0.04 cu ft/hr/ft
Concentration of lateral inflow, c_I	100 ppm
Background concentration, c_{RW}	5 ppm

Table 8.5 - Parameters for Three-Dimensional Model Test Canal.

PARAMETER	VALUE
Length, L	1600 ft
Bottom width, b	40 ft
Mean tidal depth, d_o	10 ft
Inverse side slope, left bank, s_L	0
right bank, s_R	0
Tidal amplitude, a	0.5 ft
Nikuradse's equivalent sand roughness, k	5 ft
Reach alignment angle, RANG	180 degrees
Reach decay coefficient, RDECAY	0/hr
Dimensionless dispersion coefficient	
longitudinal, K_x	0.1
lateral, K_y	0.01
vertical, K_z	0.05
Background dispersion coefficient, E_o	0 ft^2/sec
Background concentration*, c_{RW}	5 ppm
Tidal entrance decay coefficient, τ	6.21 hrs
Number of lateral layers, NLAYY	1
Number of vertical layers, NLAYZ	3
Number of saltwater interface**, DSM	5 ft
Coefficient of Equation (7.43)**, u_4	4000
Density of freshwater**, RHOF	1.94 slugs/cu ft
Density of saltwater**, RHOS	1.99 slugs/cu ft

* - unless otherwise stated
** - values for wedge analysis (Section 8.6.2)

Table 8.6 - Dimensions and Parameter Values for Network 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 diffusion, 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_o, sq ft/sec	0.0005
Vertical momentum transfer, N_z, sq ft/sec	0.002
Tidal entrance time decay, hr	1.0

Table 8.6 - continued.

Concentration:	
Pollutant inflow concentration, ppm	100.00
Pollutant inflow rate, $ft^3/ft\text{-}hr$	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

Table 8.7 - Comparison of Concentration Values in Systems A and D Under No Wind and East Wind Conditions.

		Concentration Above Background (c_{RW} in ppm) After 30 Tidal Cycles							
		NO WIND				EAST WIND			
		System A		System D		System A		System D	
		Loc.	Conc.	Loc.	Conc.	Loc.	Conc.	Loc.	Conc.
Surface:	Junctions	J2	1.019	J5	0.937	J2	0.694	J5	0.681
		J3	0.087	J4	0.081	J3	0.073	J4	0.069
		J4	0.032	J3	0.032	J4	0.032	J3	0.033
		J5	0.027	J2	0.026	J5	0.025	J2	0.026
	Dead-ends	R1	2.160	NP*	2.169	R1	1.364	NP*	1.570
		R3	2.160	R6	2.169	R3	0.864	R6	0.882
		R5	2.160	R4	2.169	R5	0.562	R4	0.573
		R7	2.160	R2	2.169	R7	0.379	R2	0.383
Bottom:	Junctions	J2	1.221	J5	1.154	J2	0.942	J5	0.932
		J3	0.395	J4	0.370	J3	0.306	J4	0.295
		J4	0.030	J3	0.025	J4	0.018	J3	0.015
		J5	0.001	J2	0.001	J5	0.001	J2	0.001
	Dead-ends	R1	2.091	NP*	2.104	R1	1.357	NP*	1.491
		R3	2.091	R6	2.104	R3	0.849	R6	0.866
		R5	2.091	R4	2.104	R5	0.541	R4	0.550
		R7	2.091	R2	2.104	R7	0.351	R2	0.354

*NP is the null point in System D.

Table 8.8 - Variability Tests on Design Elements.

System	Lake Area Ft^2	Bend Radius Ft	No. of Tidal Entrances	No. of Tidal Cycles	Wind Dir.	Run No.	Figure No.
A	None	None	1	30	None	544	8.32
A	None	None	1	50	West	141	8.33
A	None	None	1	30	East	718	8.35
B	100,000	None	1	48	None	779	8.38
B	100,000	None	1	48	West	794	8.40
B	100,000	None	1	48	East	970	8.41
C	None	2466	1	50	East	538	8.43
D	None	None	2	50	None	915	8.45
D	None	None	2	50	East	642	8.46

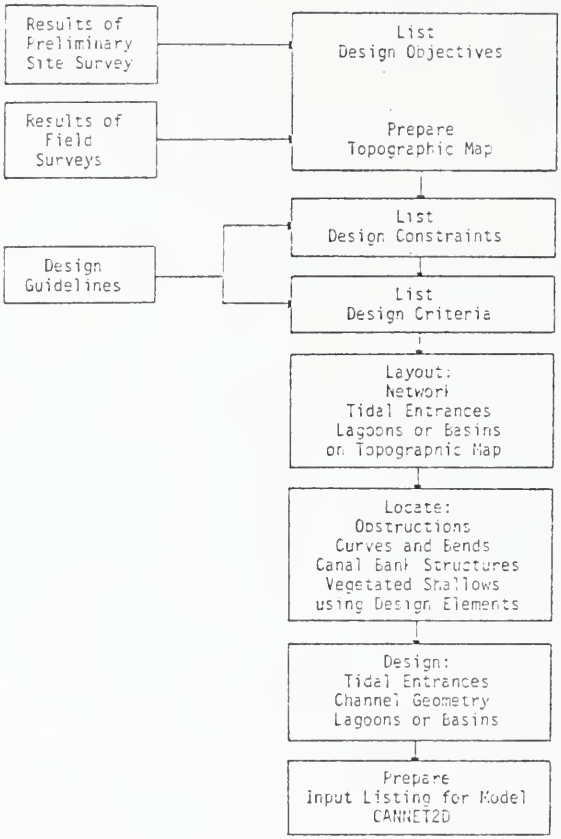


Figure 8.1 - Steps in Formulating a Trial Canal Design.

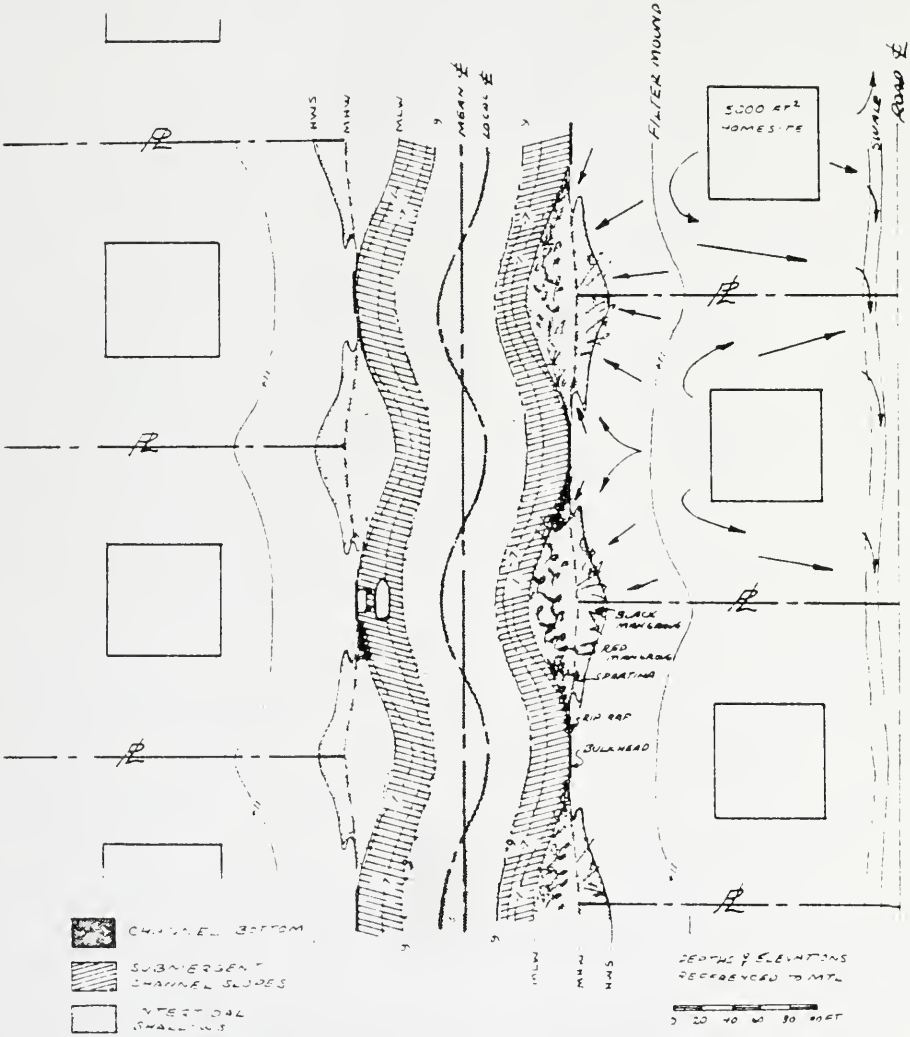


Figure 8.2 - Sinuous Bank Design for 57 Acres Project (Source: Snyder, R. M., 1976a, p. 77).

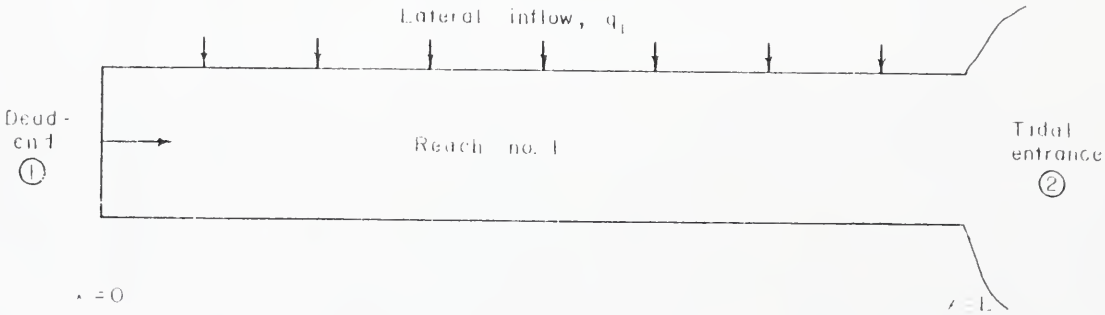


Figure 8.3 - First Test Canal Network.

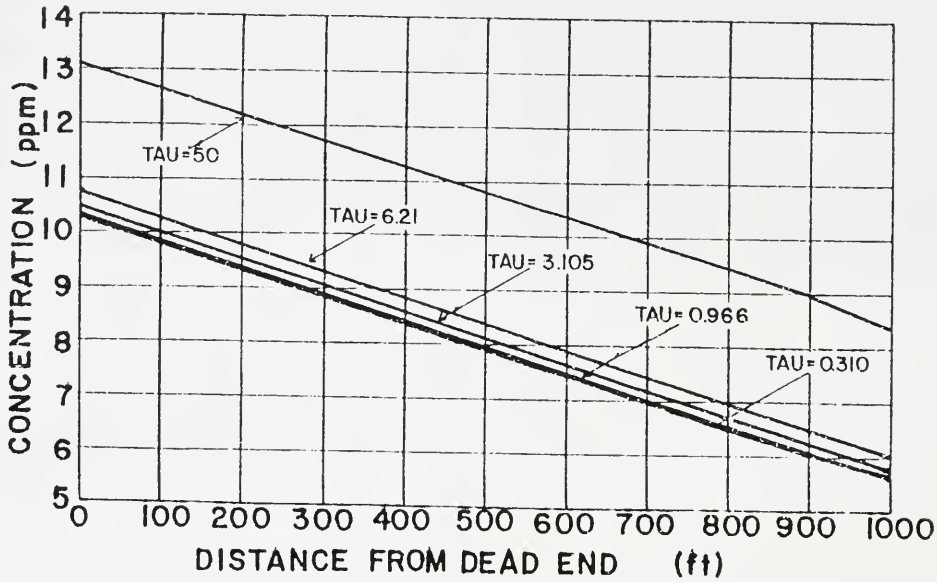


Figure 8.4 - Variability of Tidal Entrance Time Decay Coefficient, τ.

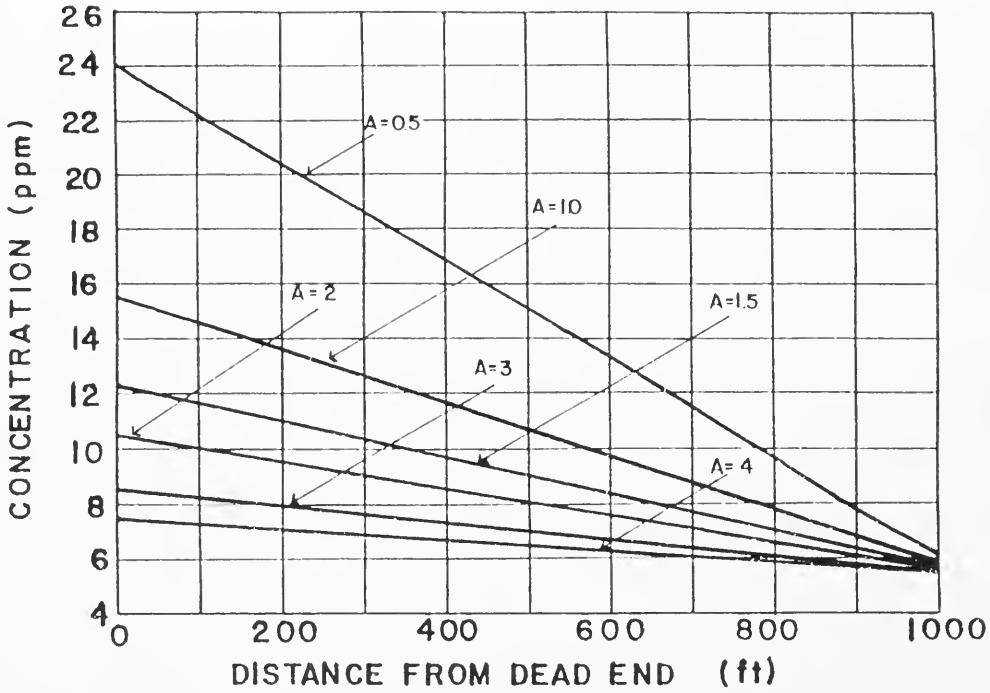


Figure 8.5 - Variability of Tidal Amplitude, a.

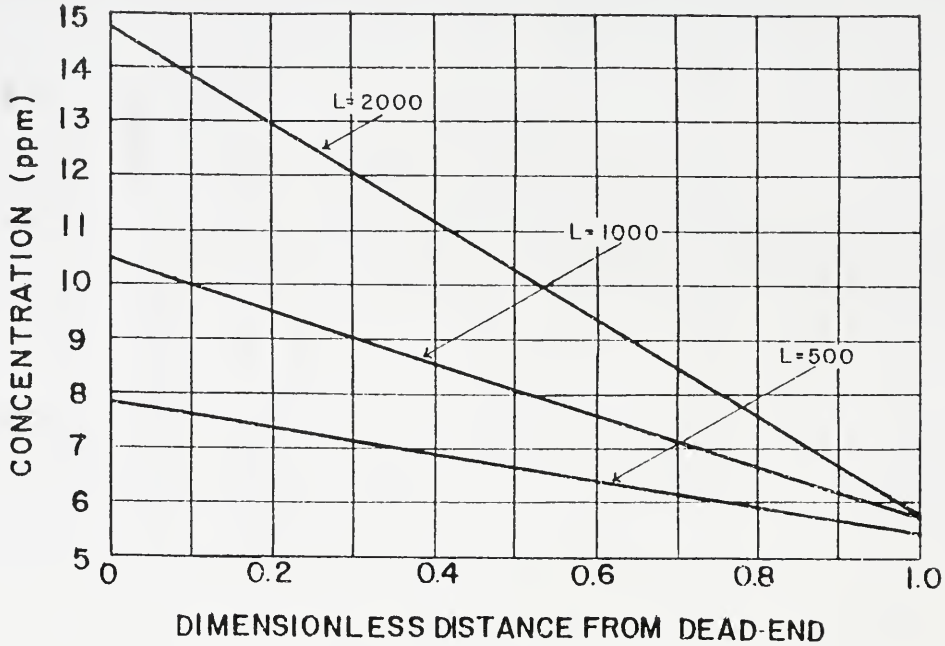


Figure 8.6 - Variability of Canal Length, L.

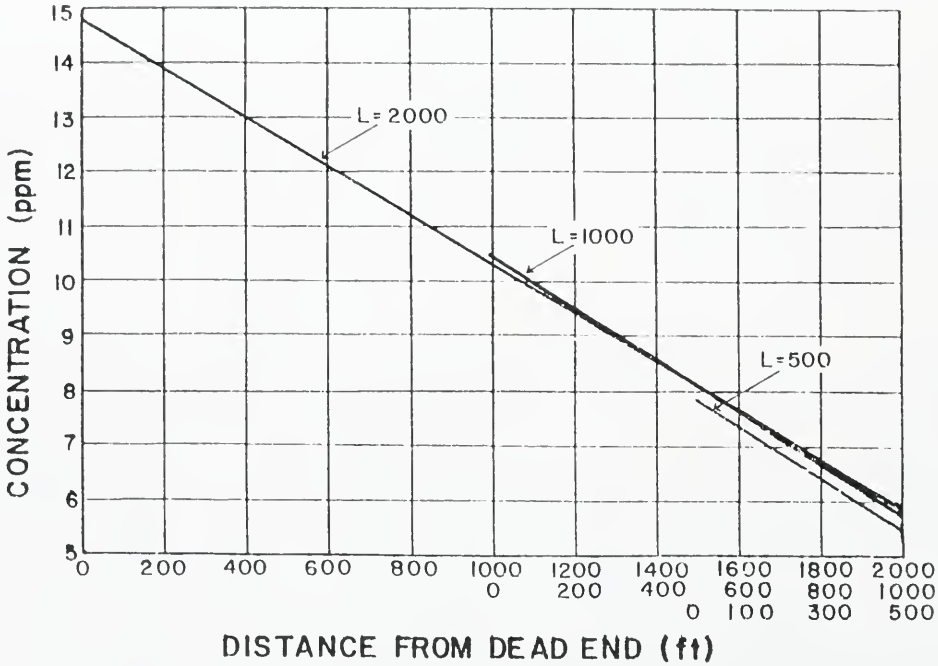


Figure 8.7 - Variability of Dimensionless Canal Length.

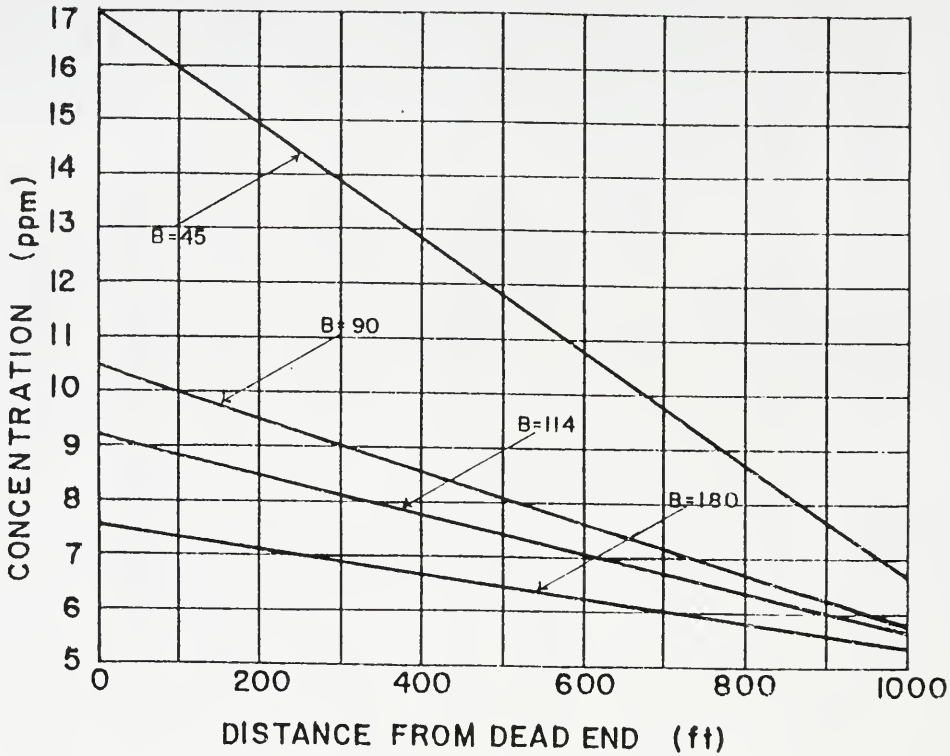


Figure 8.8. Variability of Bottom Width, b.

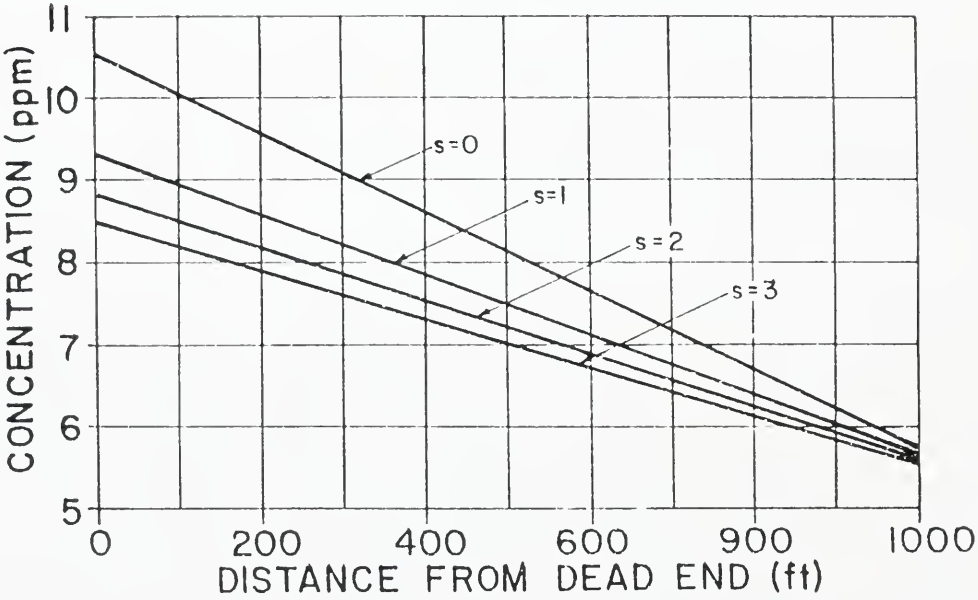


Figure 8.9 - Variability of Inverse Side Slope, s.

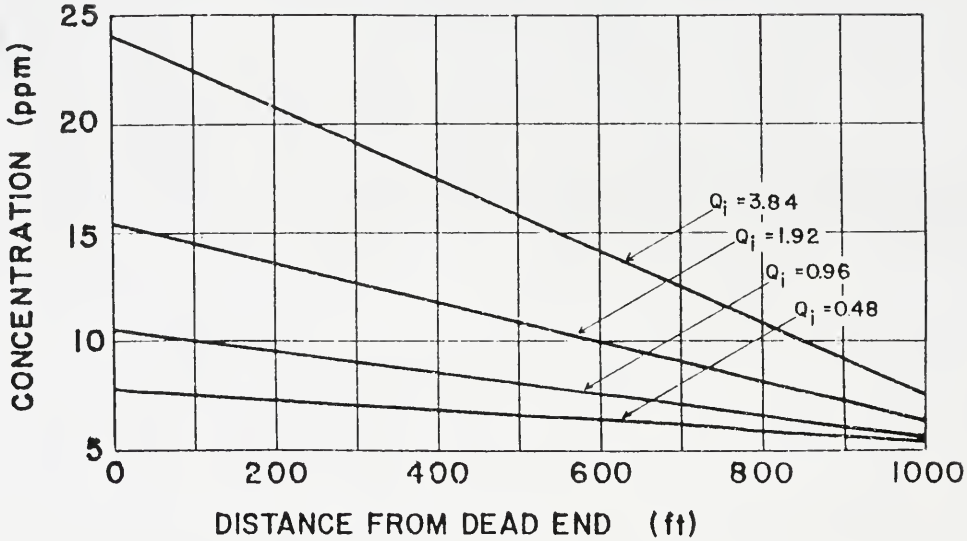


Figure 8.10 - Variability of Lateral Inflow Rate, q_I.

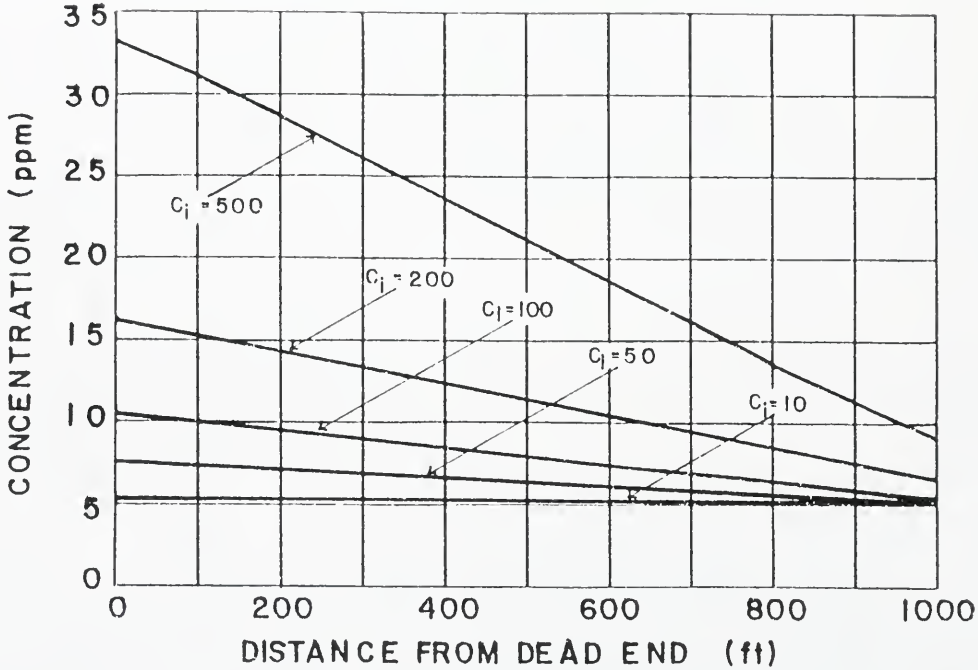


Figure 8.11 - Variability of Lateral Inflow Concentration, c_I.

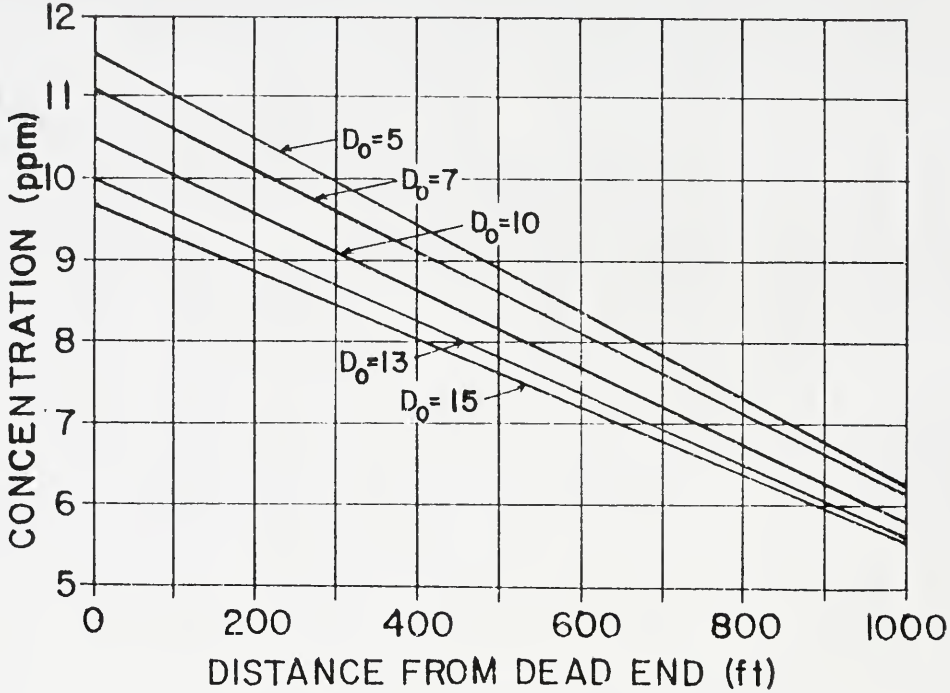


Figure 8.12 - Variability of Mean Tidal Depth, d_o.

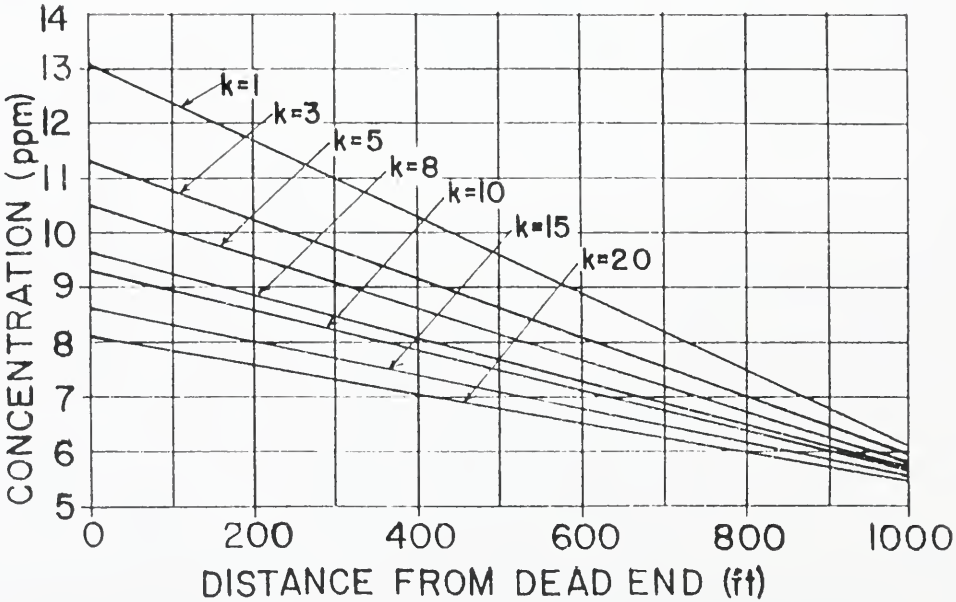


Figure 8.13 - Variability of Nikuradse's Equivalent Sand Roughness, k.

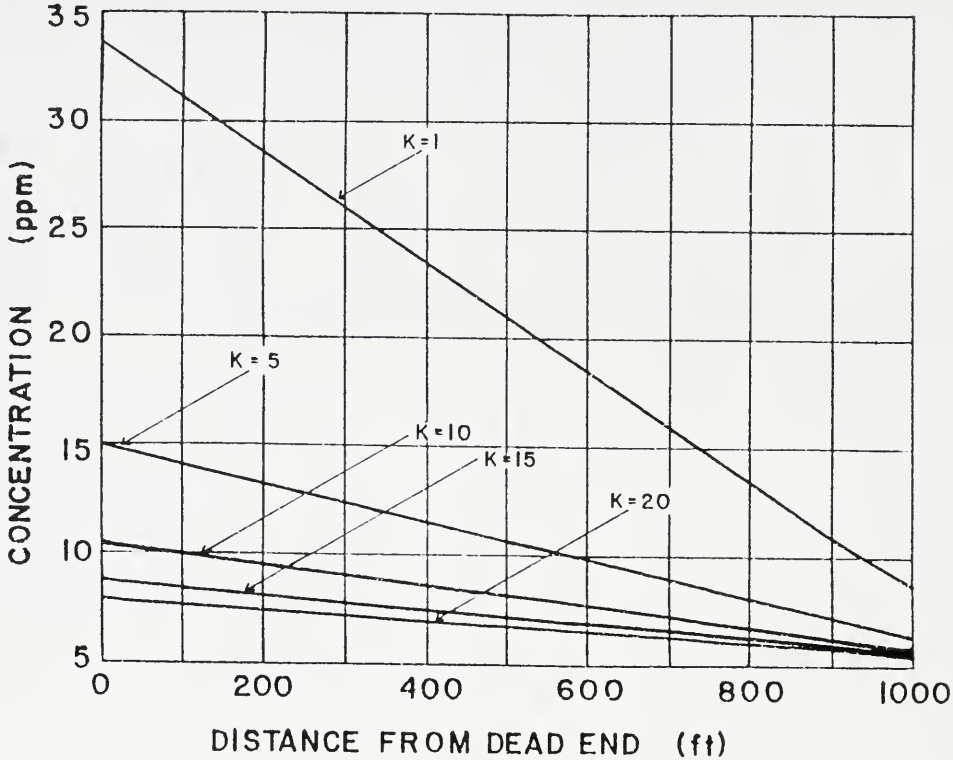


Figure 8.14 - Variability of Dimensionless Dispersion Coefficient, K.

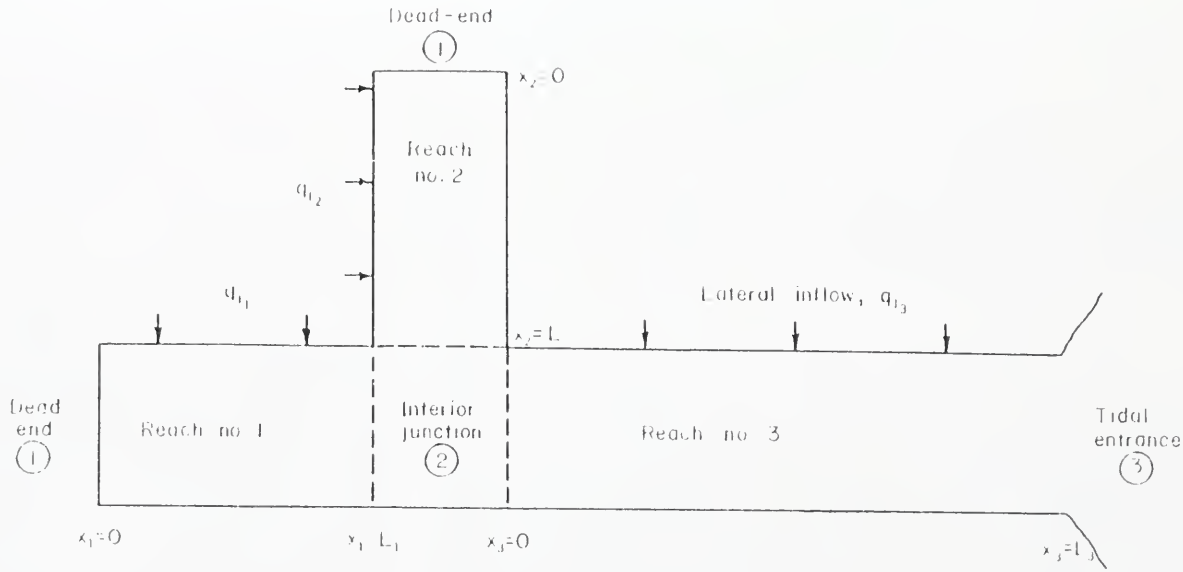


Figure 8.15 - Second Test Canal Network.

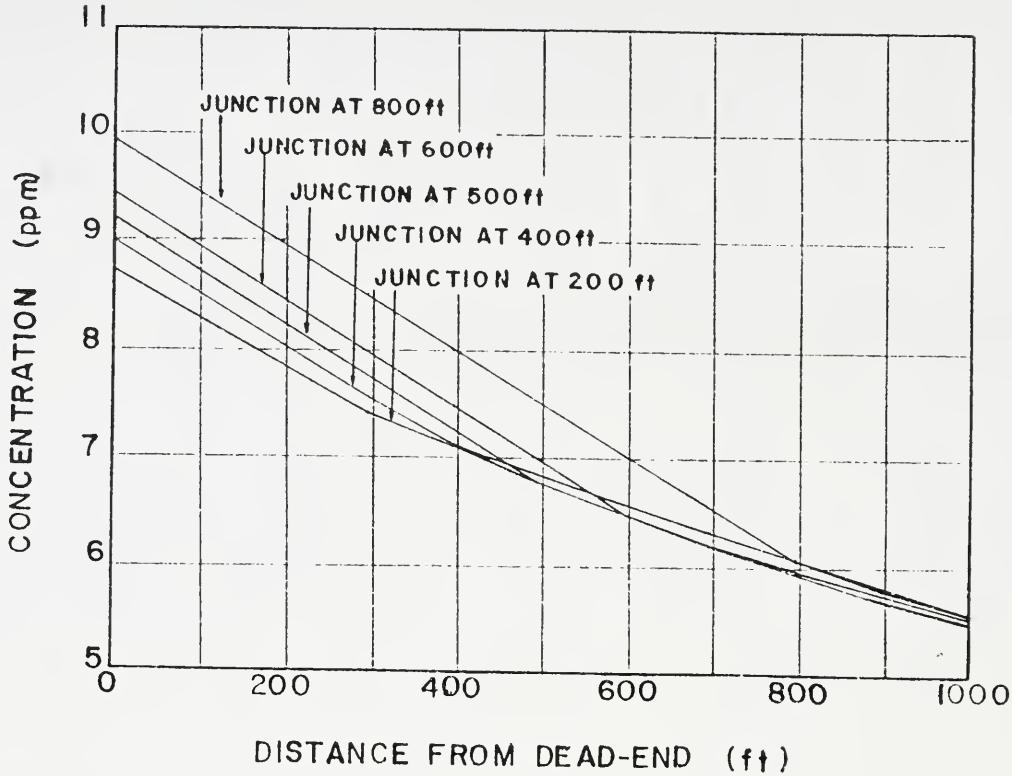


Figure 8.16 - Variability of Low Tide Concentration Profiles for Various Branch Canal Locations.

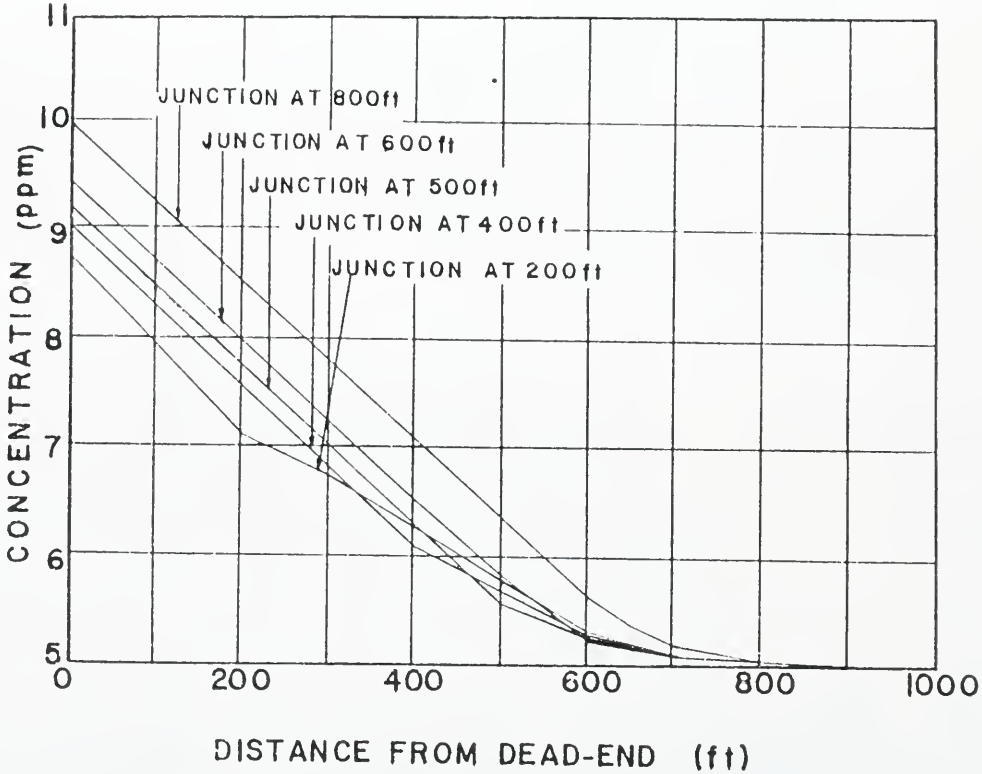


Figure 8.17 - Variability of High Tide Concentration Profiles for Various Branch Canal Locations.

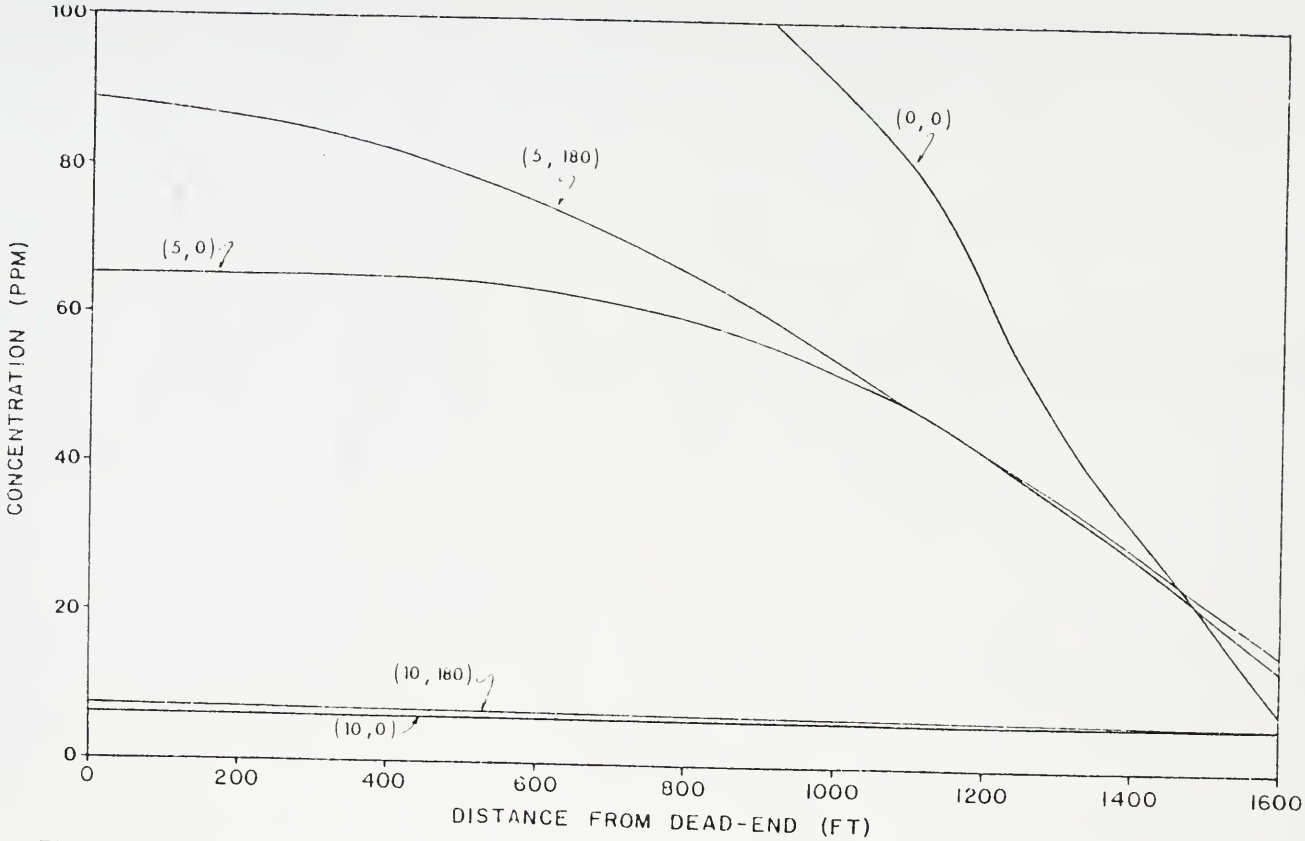


Figure 8.18 - Case 1W: Initial Concentrations, c_i = 100 ppm, Background Concentration, c_{RW} = 5 ppm - Fifty Tidal Cycles.

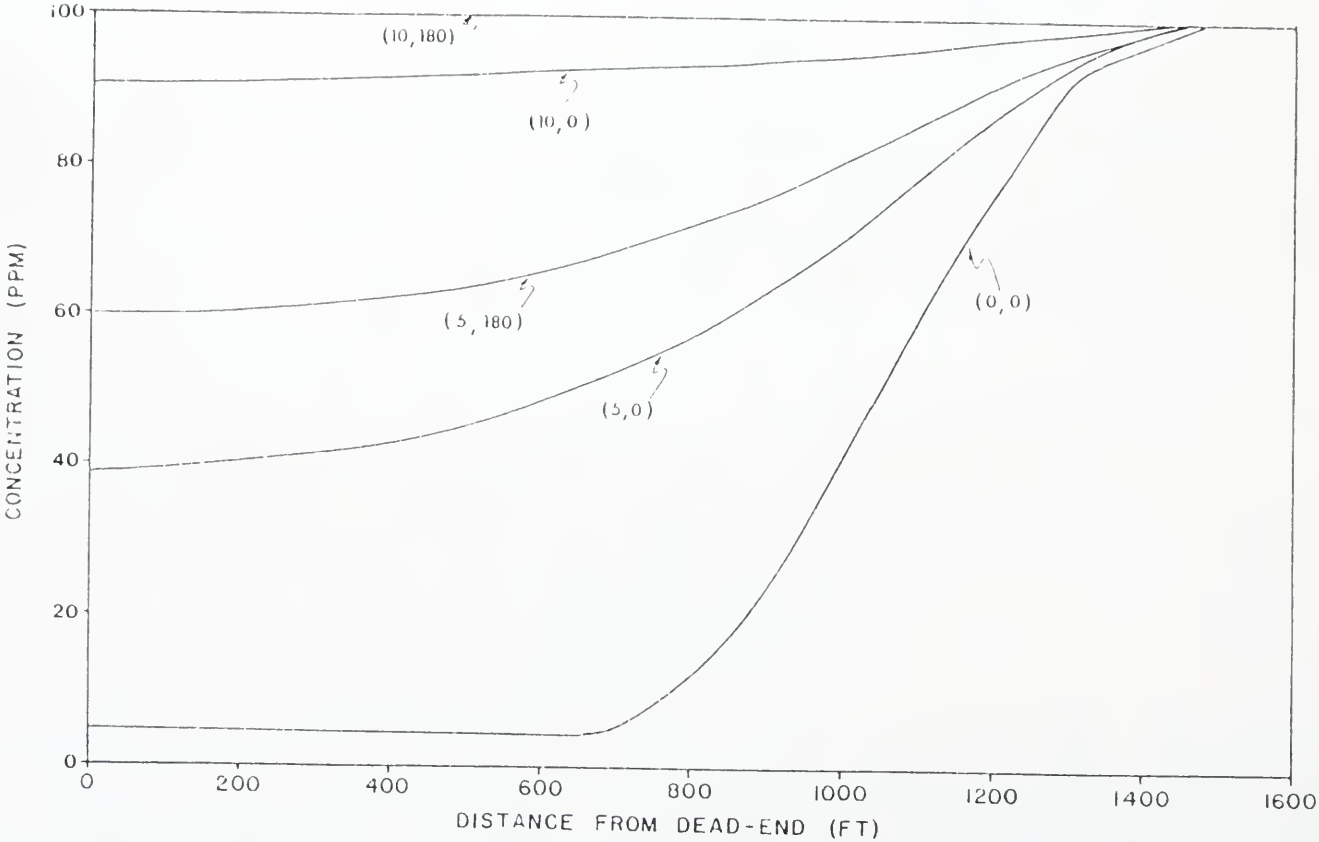


Figure 8.19 - Case 2W: Initial Concentrations, c_i = 5 ppm, Background Concentration, c_{RW} = 100 ppm - Fifty Tidal Cycles.

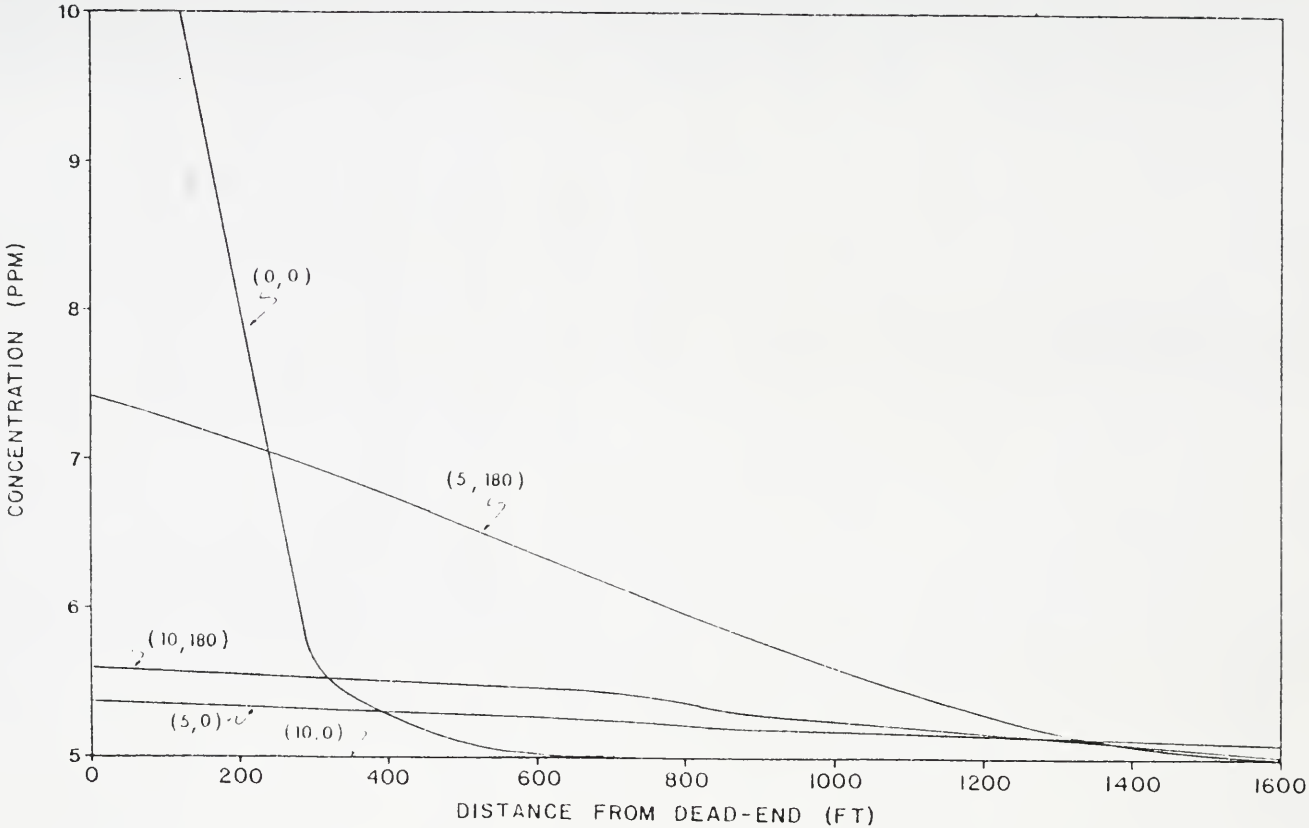


Figure 8.20 - Case 3W: Lateral Inflow Distribution Along Length of Canal - Fifty Tidal Cycles.

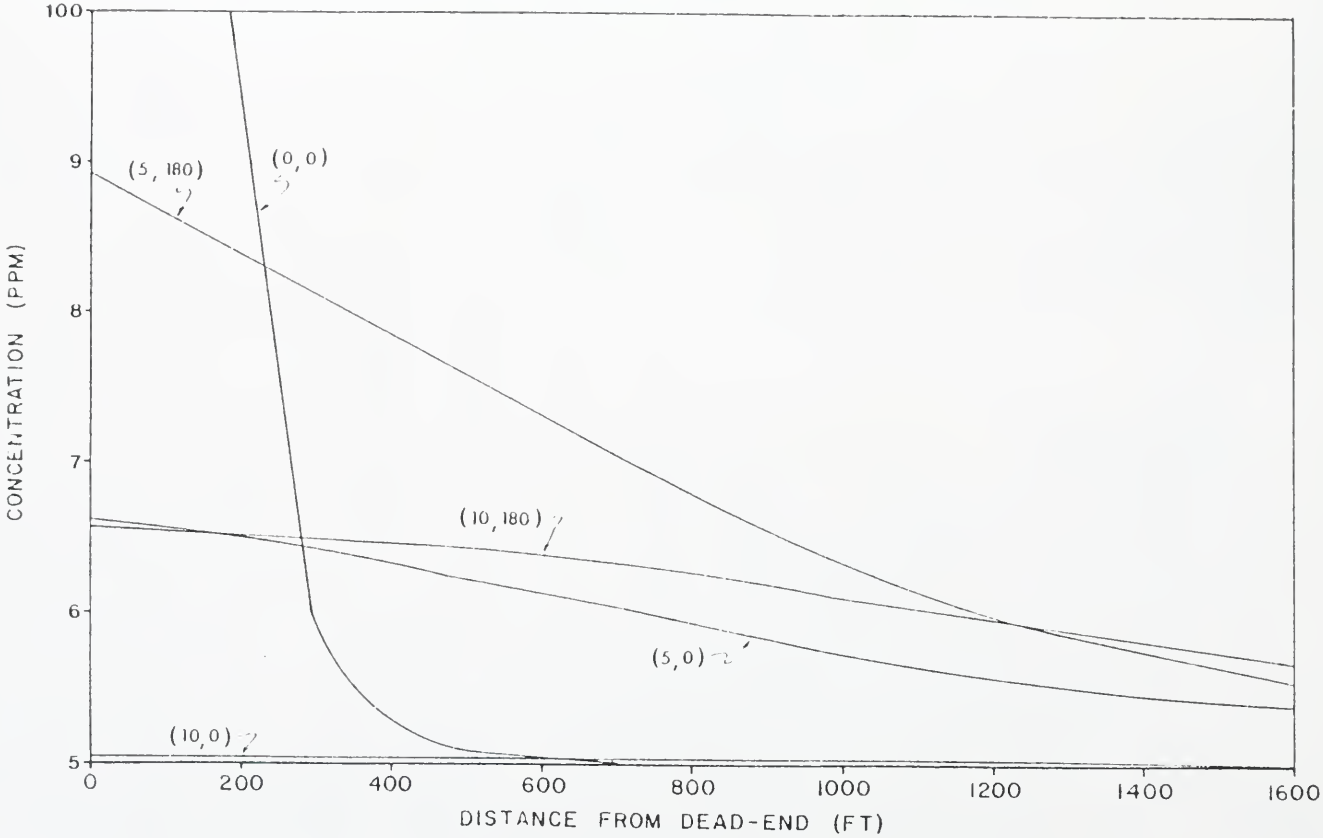


Figure 8.21 - Case 4W: Lateral Inflow Distribution Along Upper Half of Canal - Fifty Tidal Cycles.

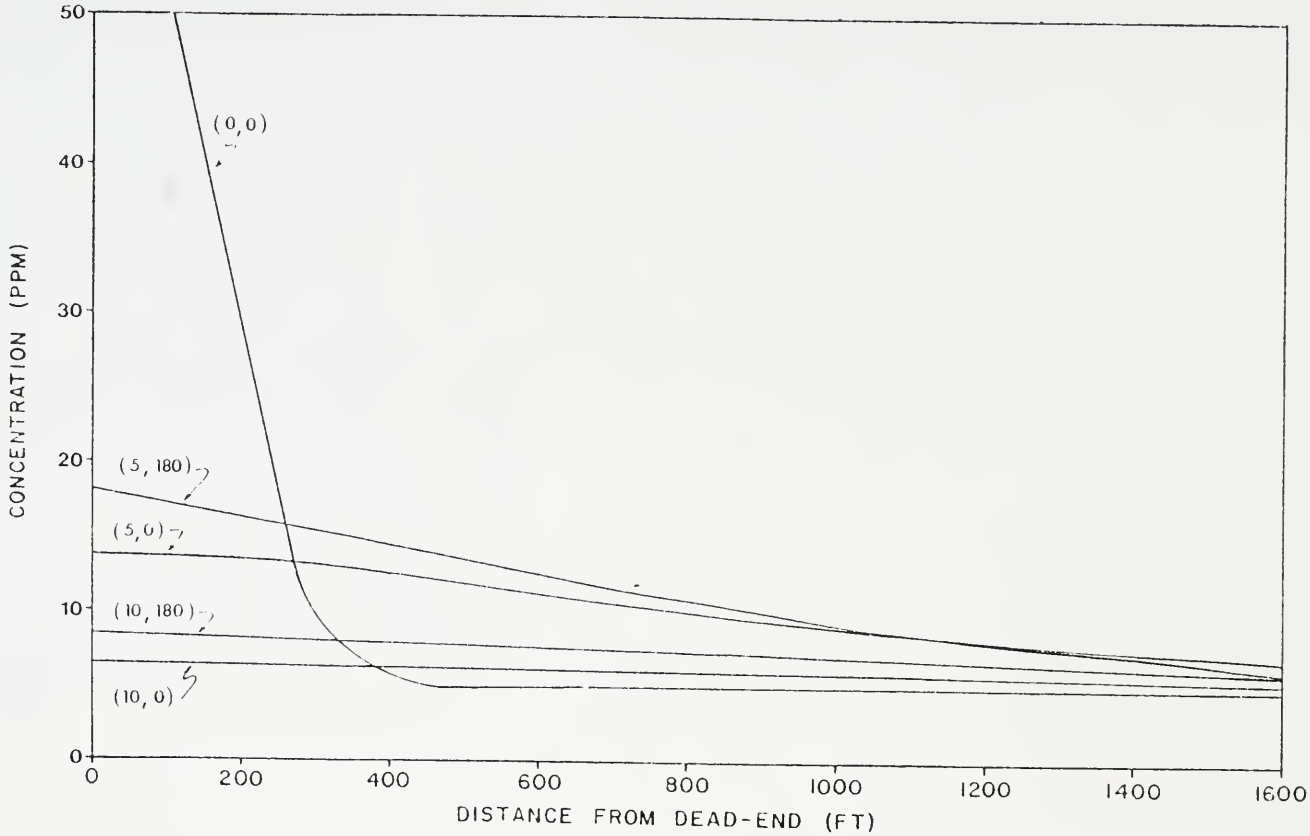


Figure 8.22 - Case 5W: Lateral Inflow at Dead-End - Fifty Tidal Cycles.

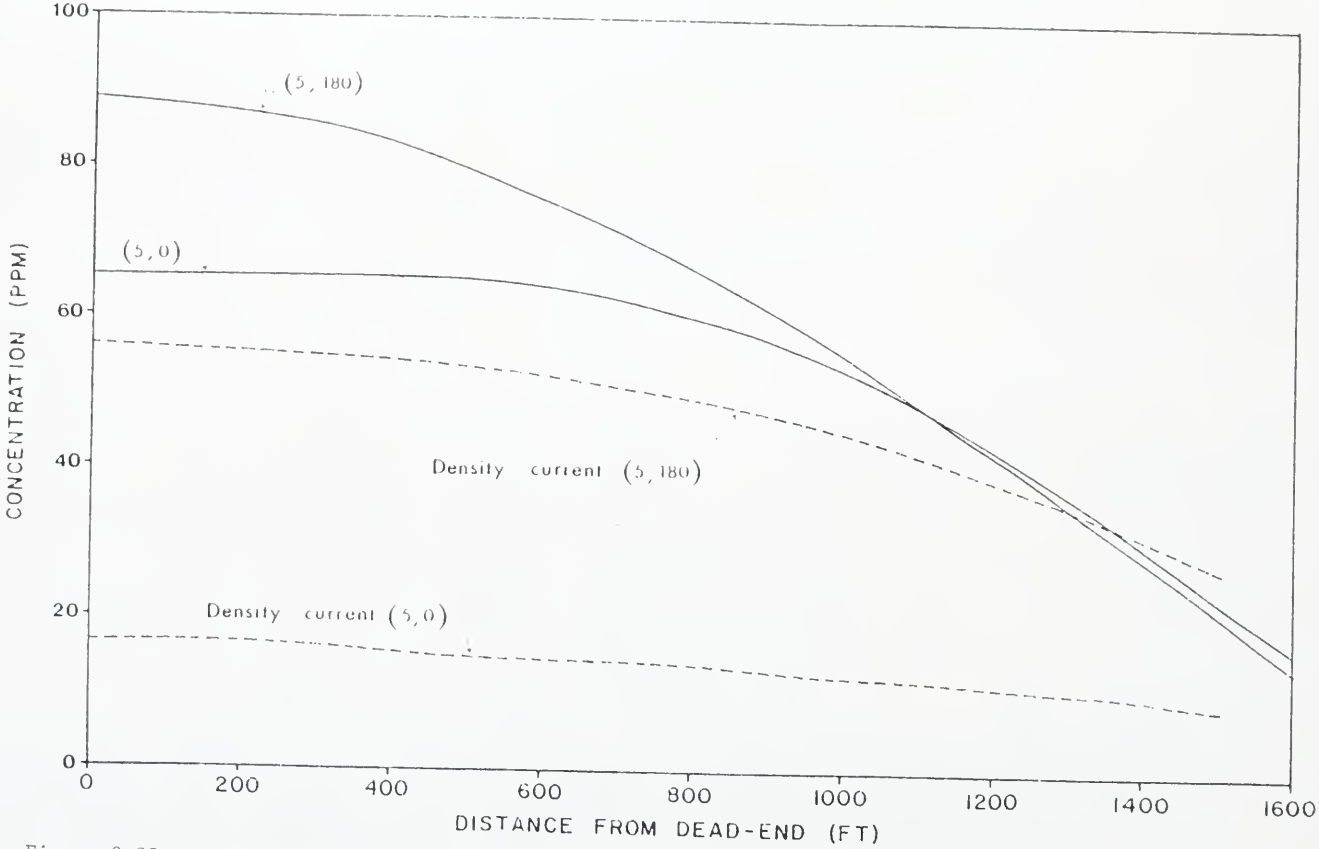


Figure 8.23 - Case 1S: Effect of Salt Wedge With Initial Concentration c_i = 100 ppm, Background Concentration, c_{RW} = 5 ppm - Fifty Tidal Cycles.

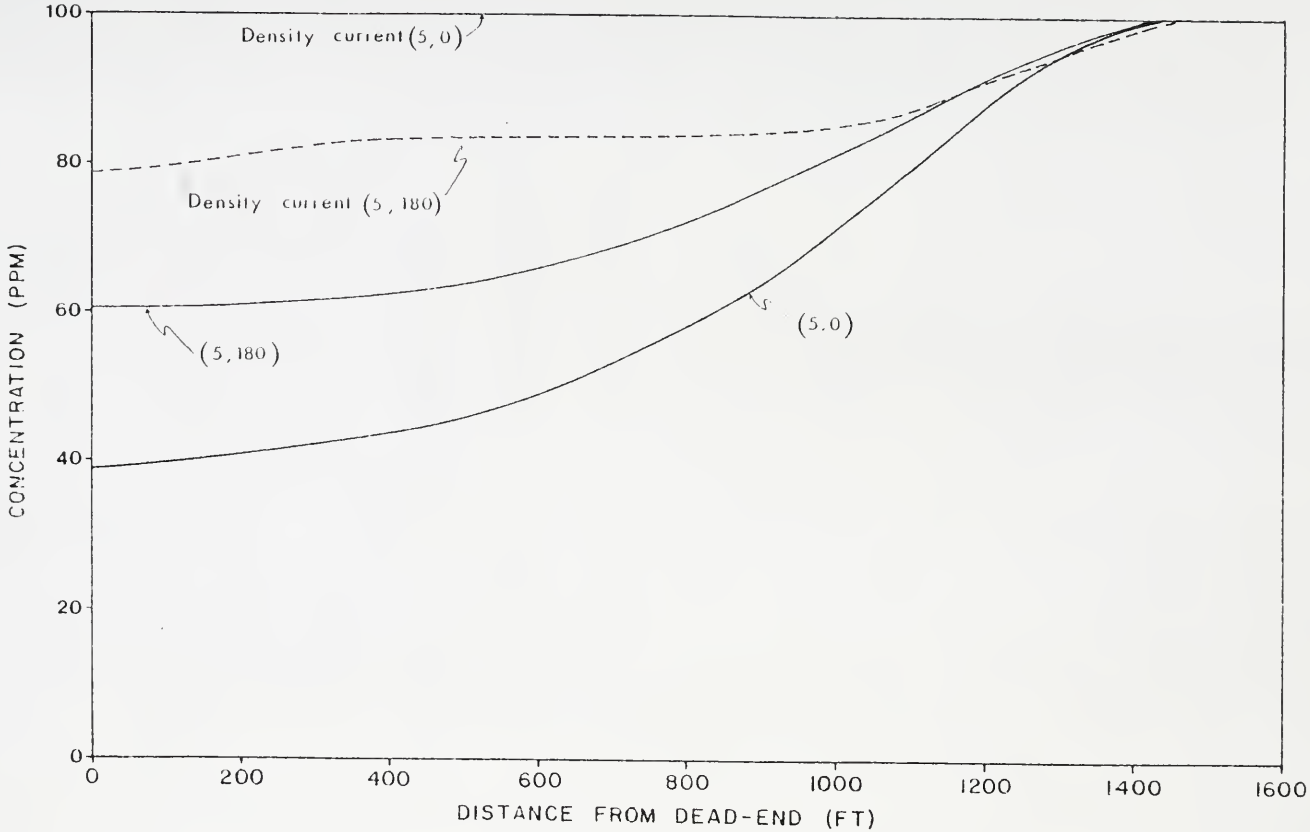


Figure 8.24 - Case 2S: Effects of Salt Wedge With Initial Concentration, c_i = 5 ppm, Background Concentration, c_{RW} = 100 ppm - Fifty Tidal Cycles.

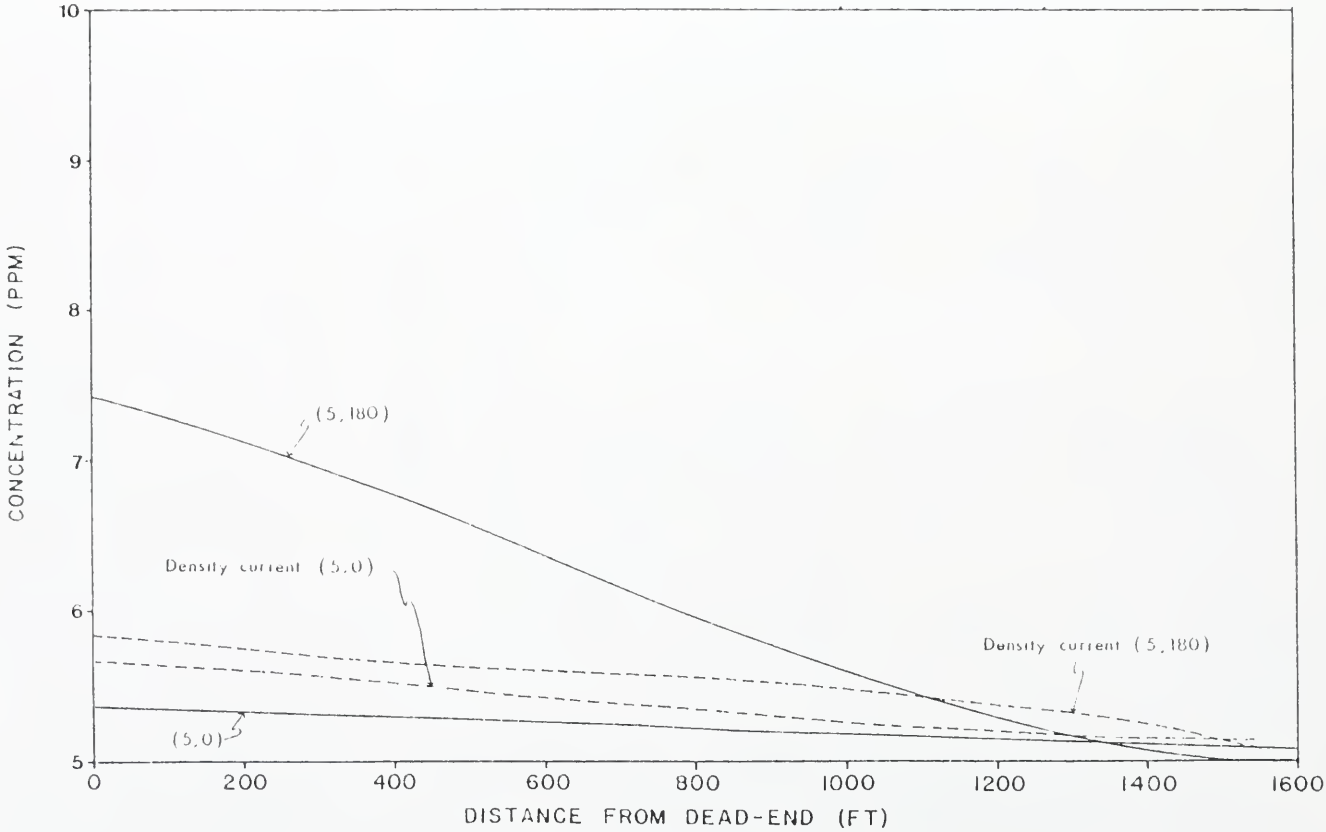


Figure 8.25 - Case 3S: Effect of Salt Wedge With Lateral Inflow Distribution Along Length of Canal - Fifty Tidal Cycles.

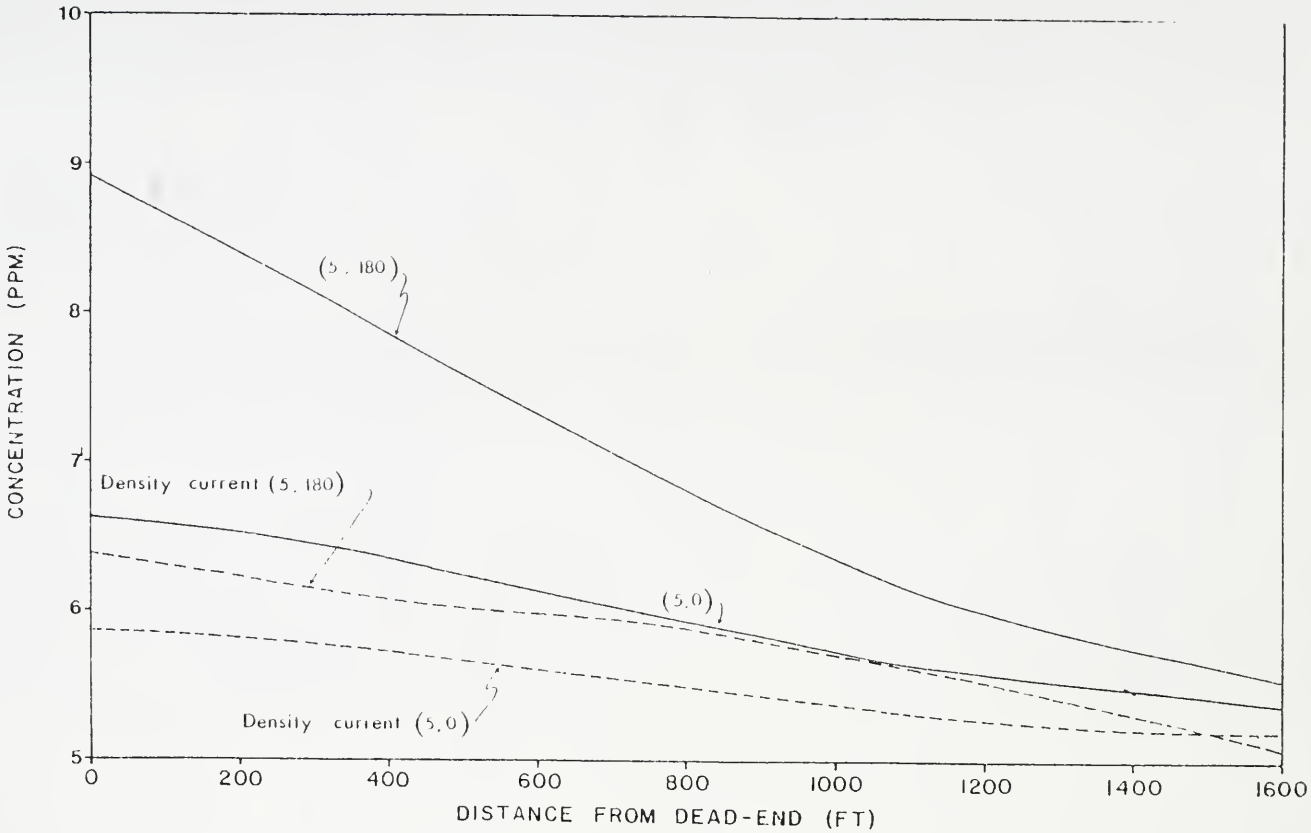


Figure 8.26 - Case 4S: Effect of Salt Wedge With Lateral Inflow Distribution Along Upper 1/2 of Canal - Fifty Tidal Cycles.

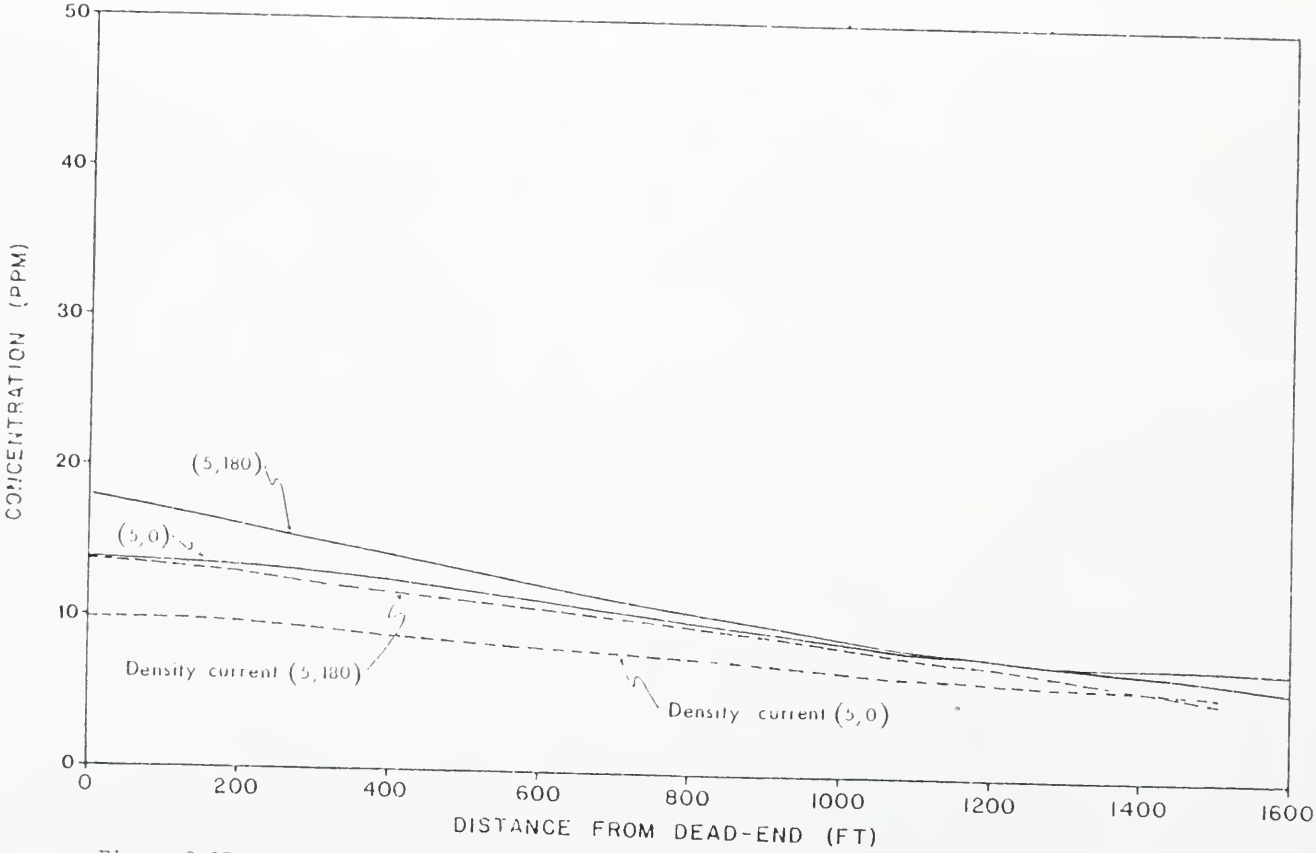


Figure 8.27 - Case 5S: Effect of Salt Wedge With Lateral Inflow Distribution at Dead-End - Fifty Tidal Cycles.

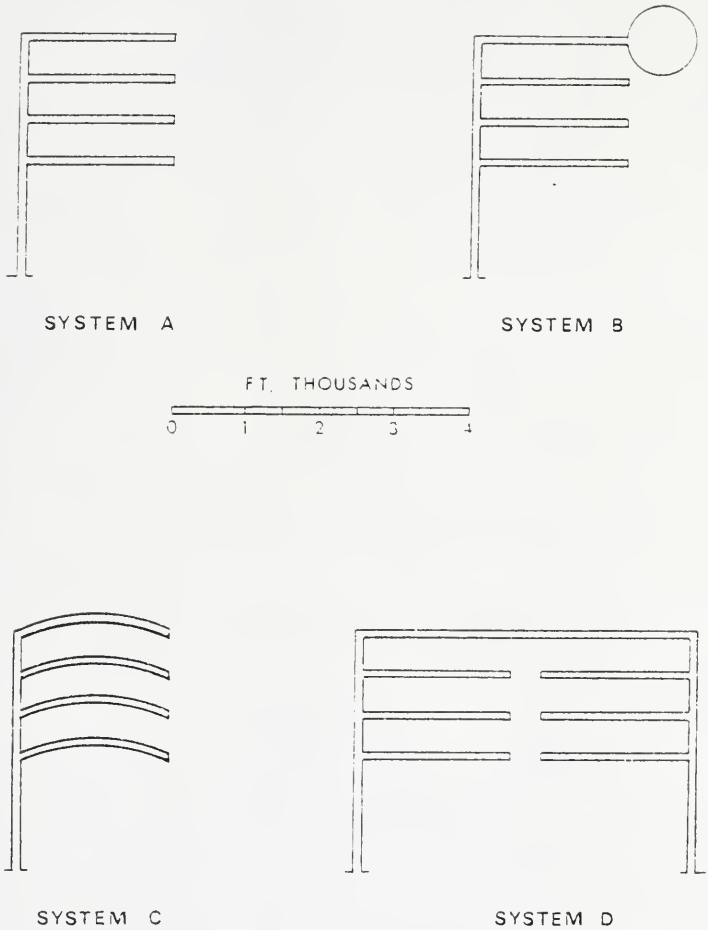


Figure 8.28 - Four Simple Network Design Elements Tested With the Mass-Transfer Model, CANNET3D.

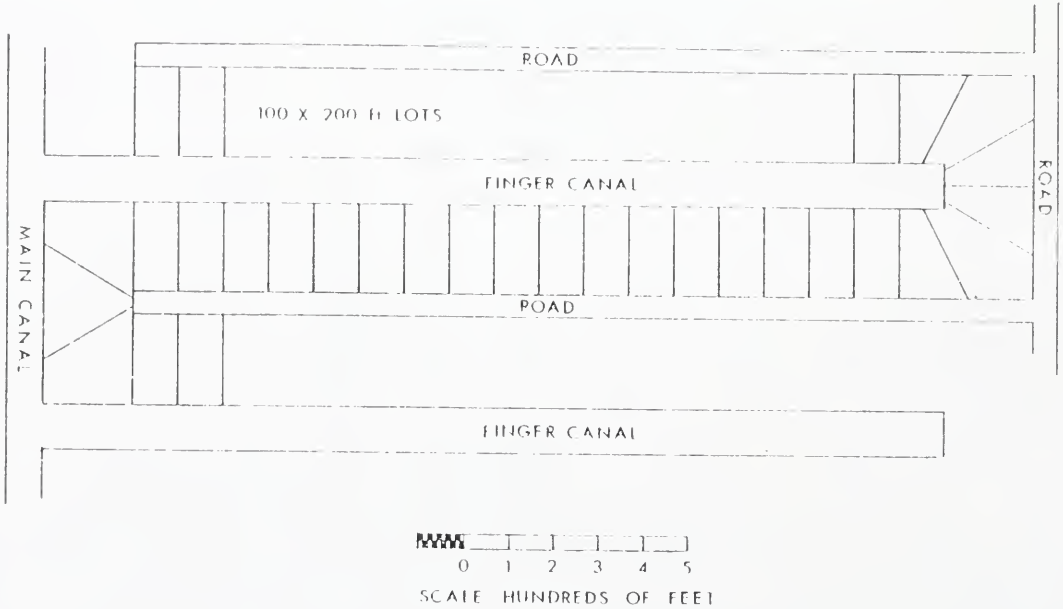


Figure 8.29 - Test Canal and Lot Dimensions.

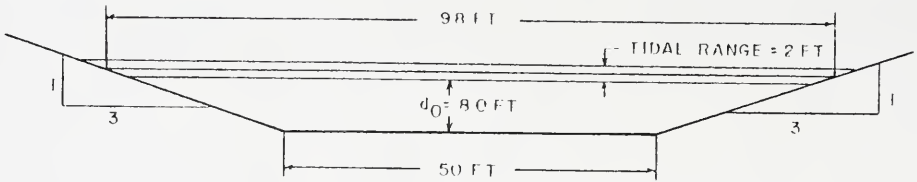


Figure 8.30 - Test Canal Cross-Section.

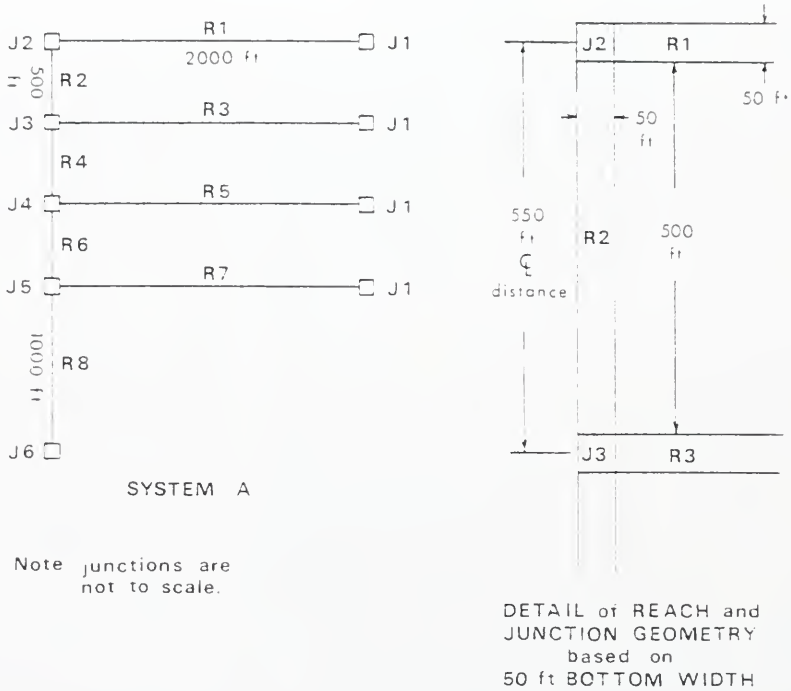


Figure 8.31 - Layout and Dimensions of System A.

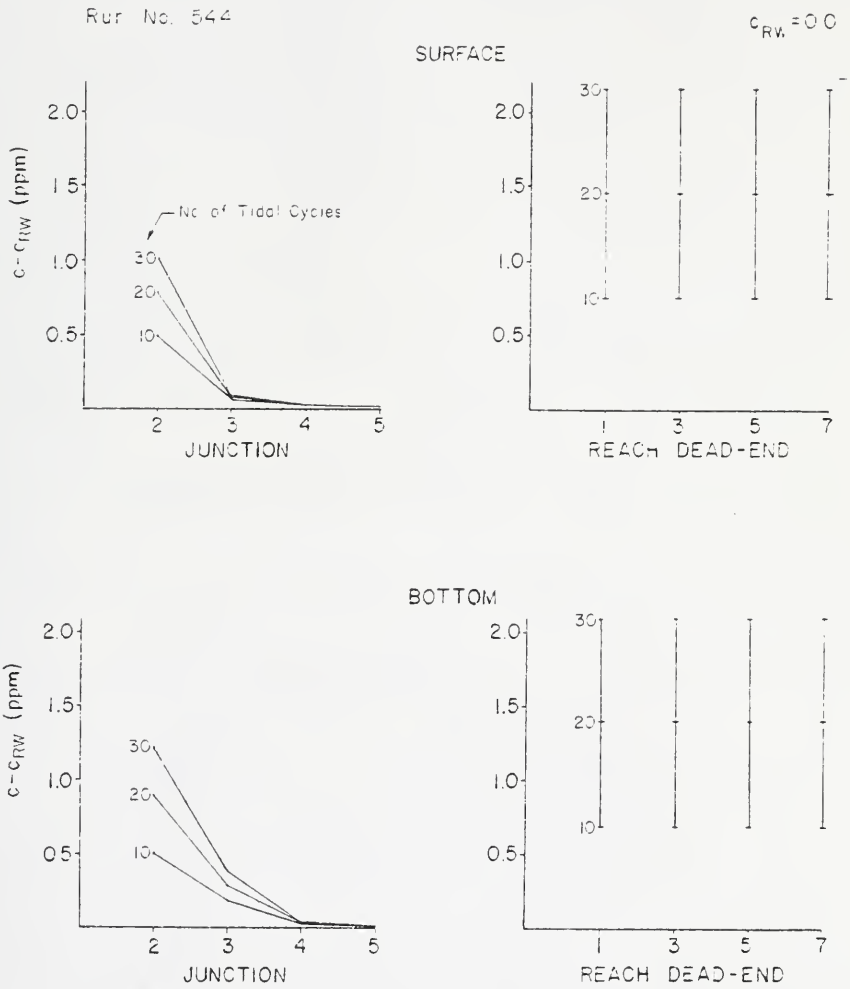


Figure 8.32 - Values of Surface and Bottom Concentrations for No Wind at Junctions and Dead-Ends at High Tide After Thirty Tidal Cycles, System A.

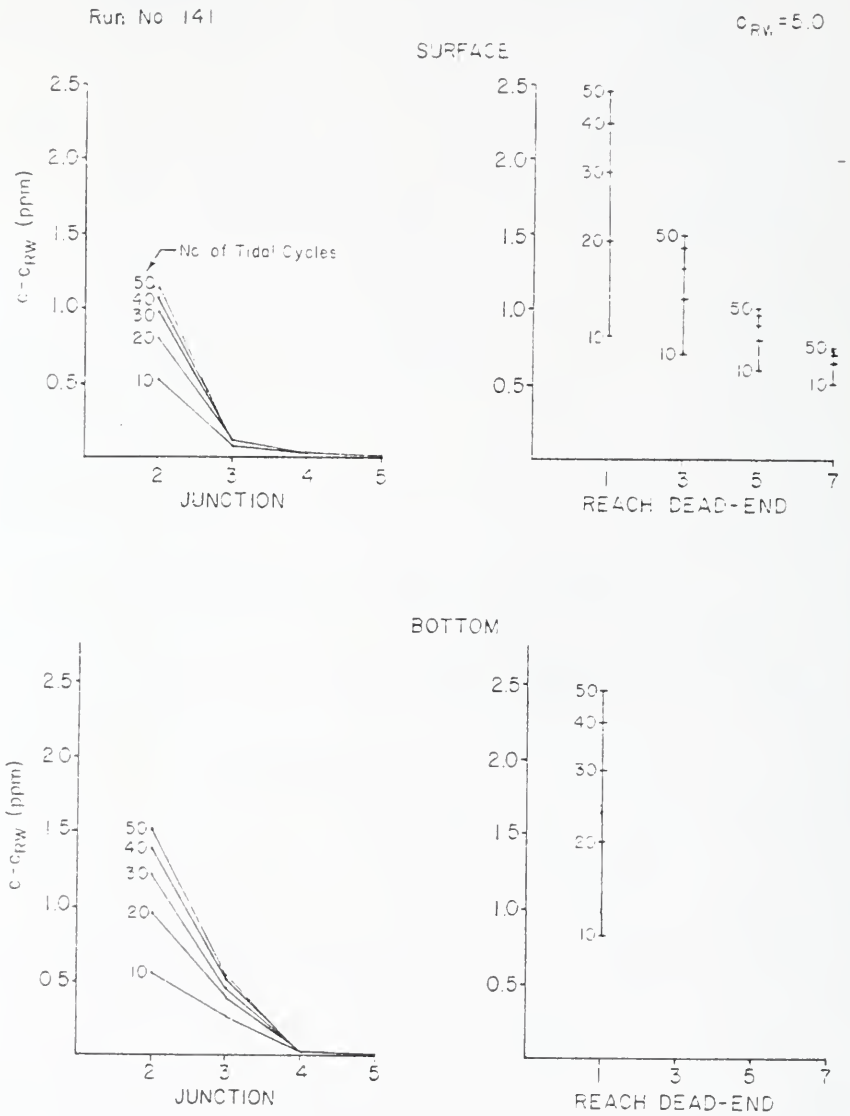
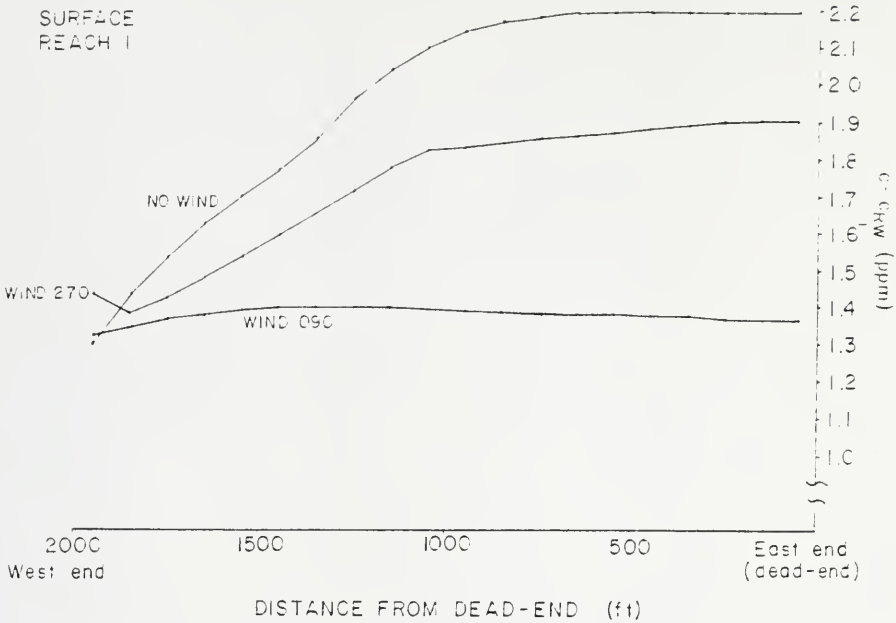


Figure 8.33 - Values of Surface and Bottom Concentration for West Wind, at Junctions and Dead-Ends at High Tide After Fifty Tidal Cycles, System A.



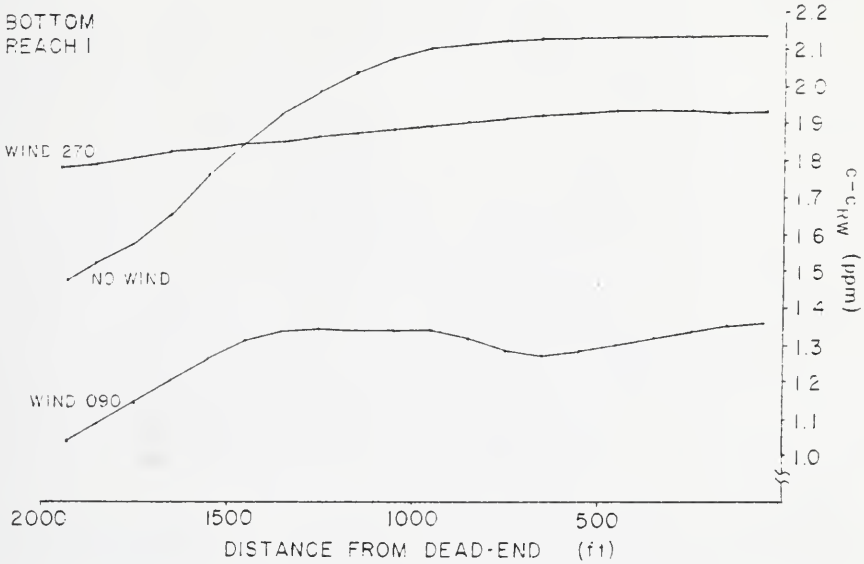


Figure 8.34 - Values of Surface and Bottom Concentration for Three Wind Conditions in Reach Number 1 at High Tide After Thirty Tidal Cycles, System A.

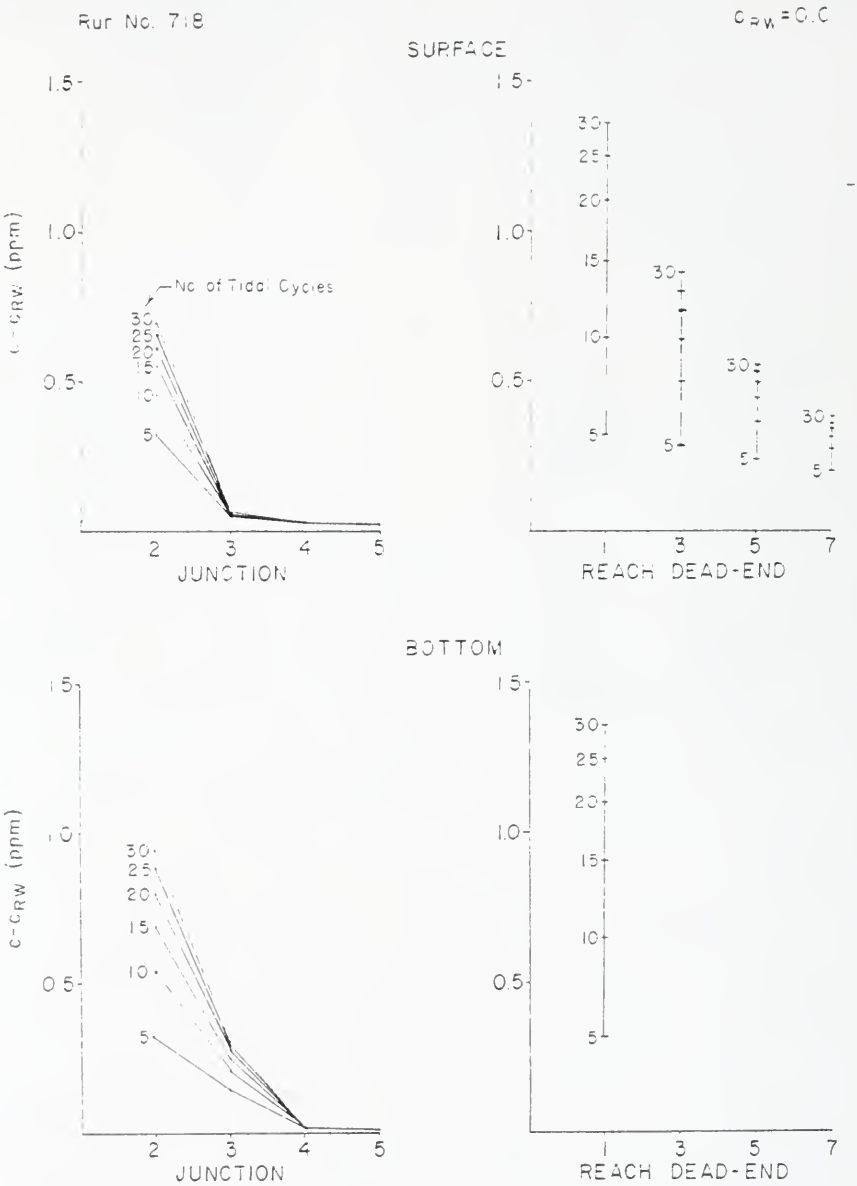


Figure 8.35 - Values of Surface and Bottom Concentration for East Wind, at Junctions and Dead-Ends at High Tide After Thirty Tidal Cycles, System A.

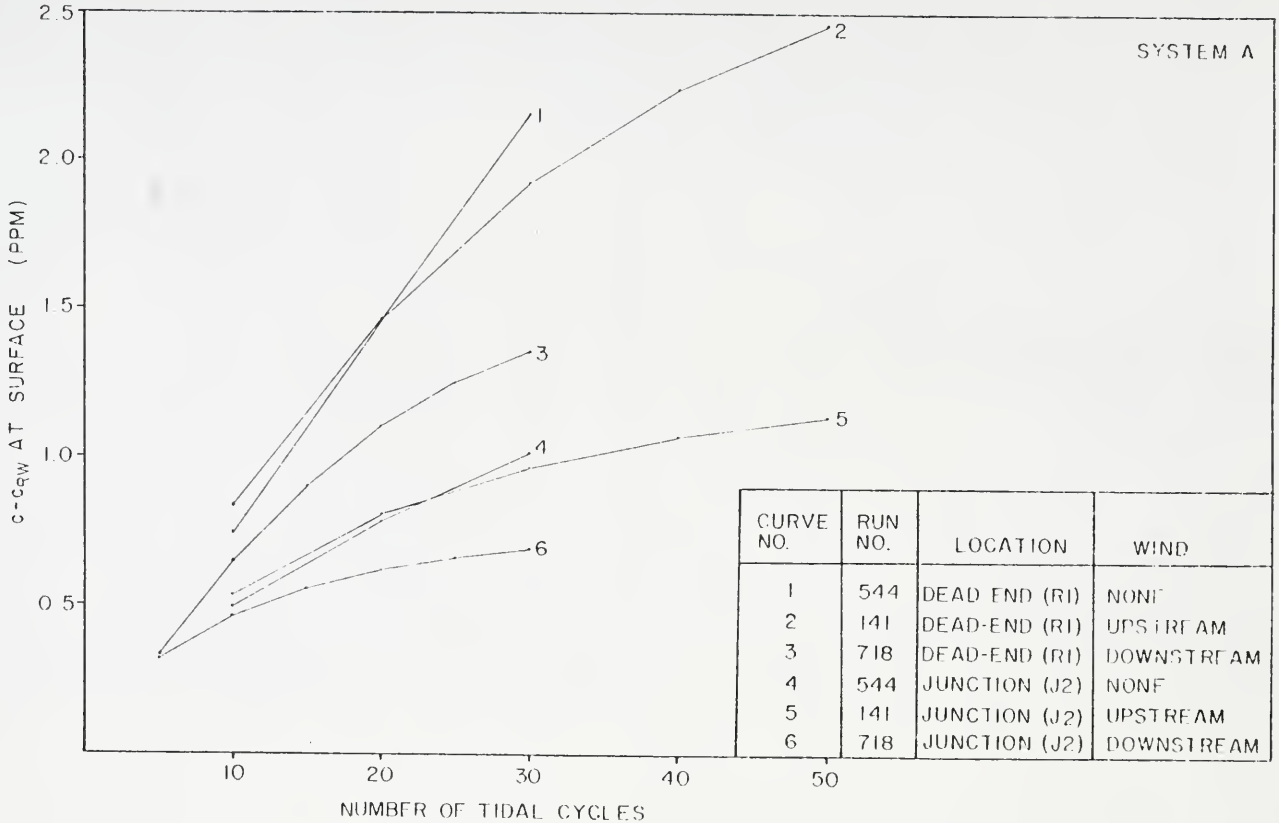


Figure 8.36 - Values of Surface Concentrations for Three Wind Conditions at Dead-End of Reach Number 1 and Junction 2 Versus Number of Tidal Cycles From Beginning of Simulation, System A.

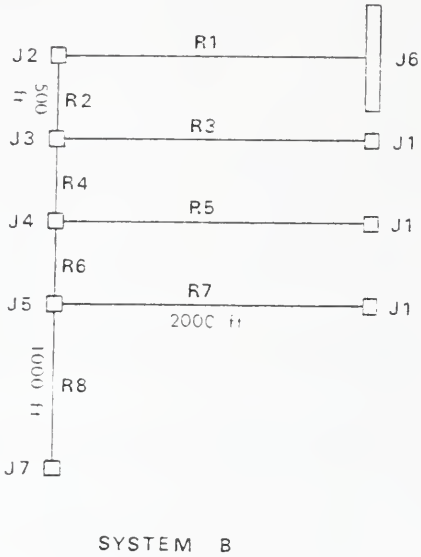


Figure 8.37 - Layout and Dimensions of System B.

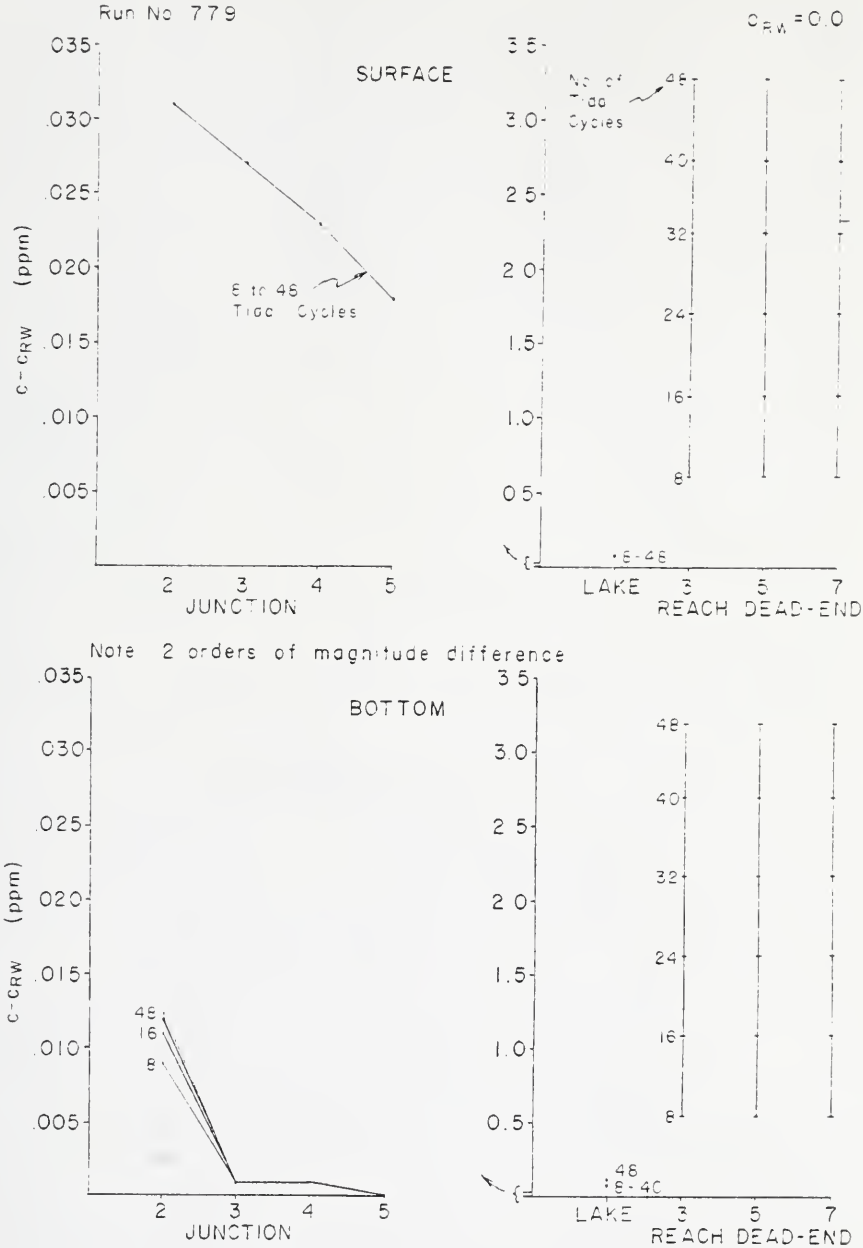


Figure 8.38 - Values of Surface and Bottom Concentrations for No Wind at Junctions and Dead-Ends at High Tide After Forty-Eight Tidal Cycles, System B.

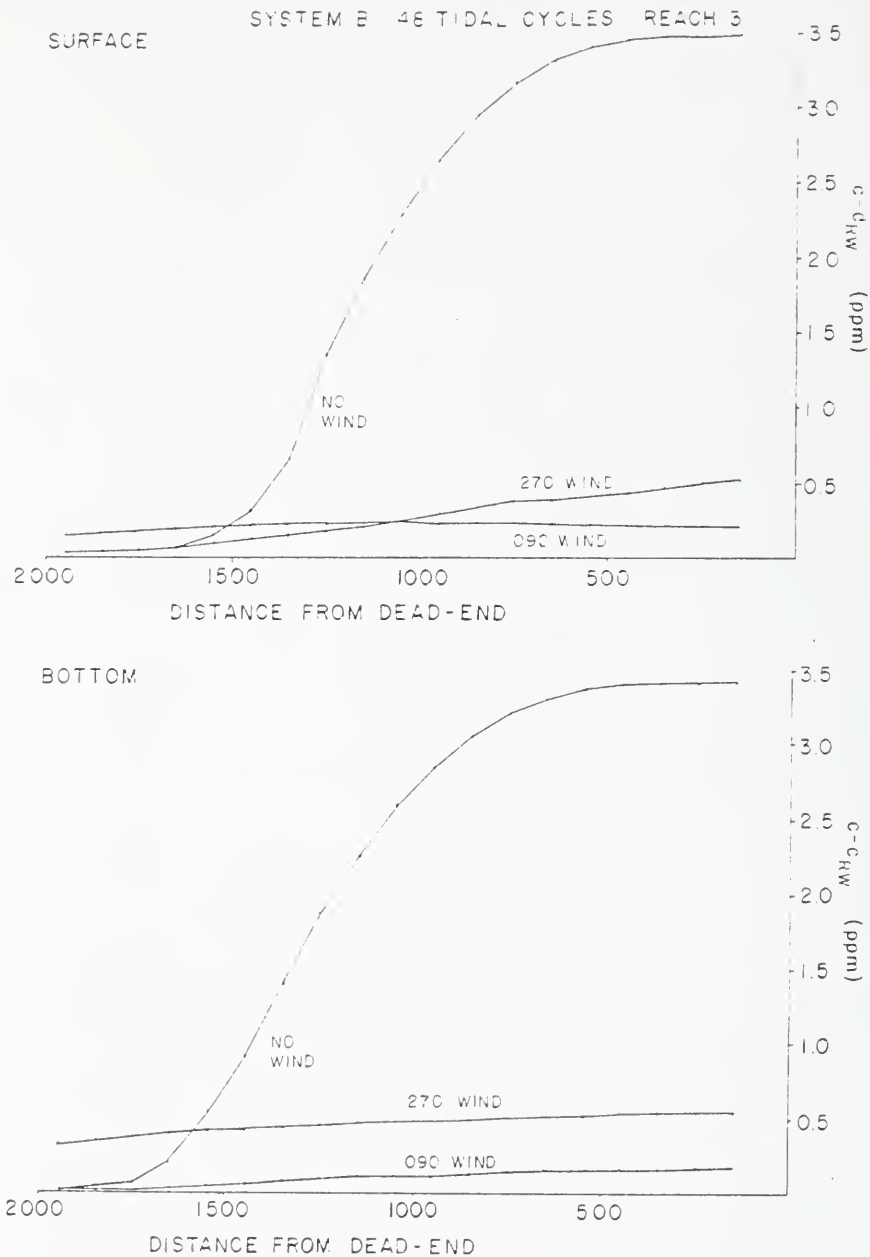


Figure 8.39 - Values of Surface and Bottom Concentrations for Three Wind Conditions in Reach Number 3 at High Tide After Forty-Eight Tidal Cycles, System B.

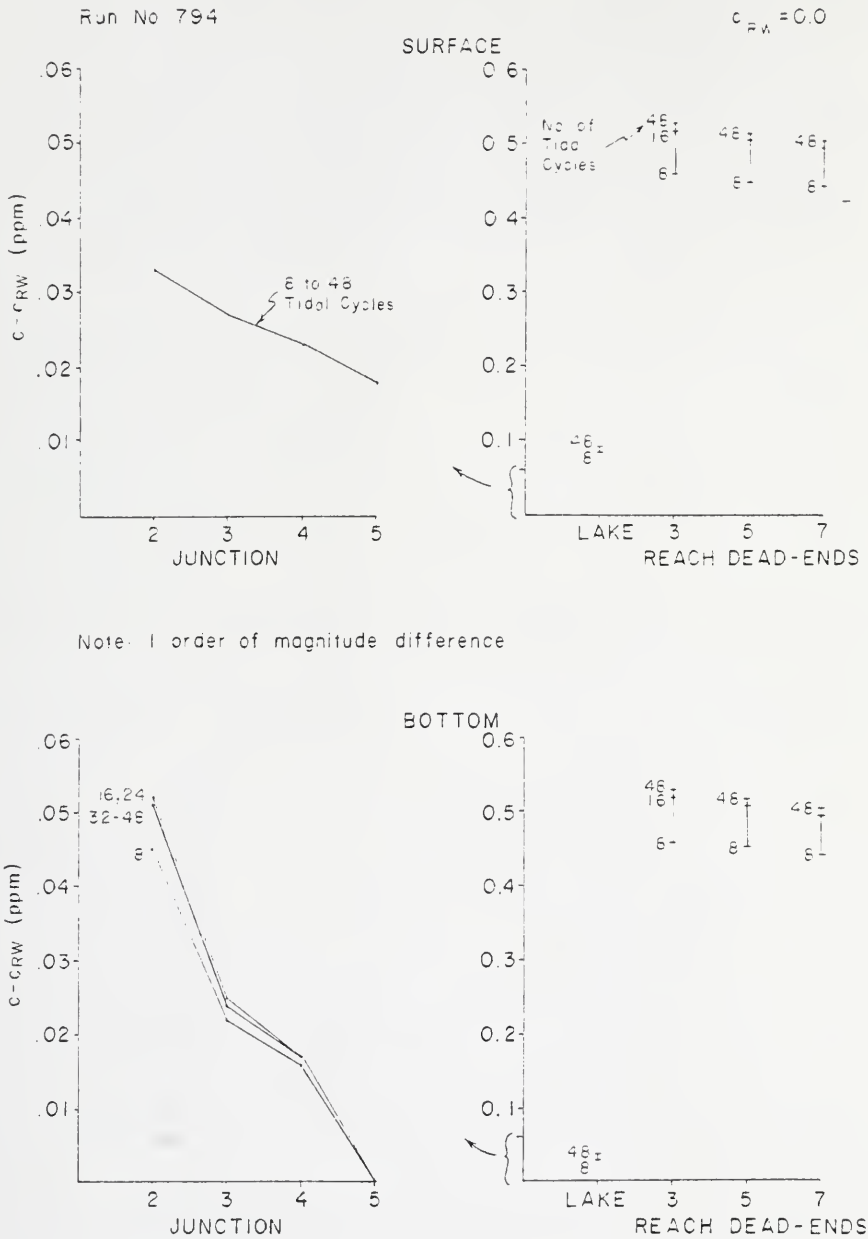


Figure 8.40 - Values of Surface and Bottom Concentrations for West Wind at Junctions and Dead-Ends at High Tide After Forty-Eight Tidal Cycles, System B.

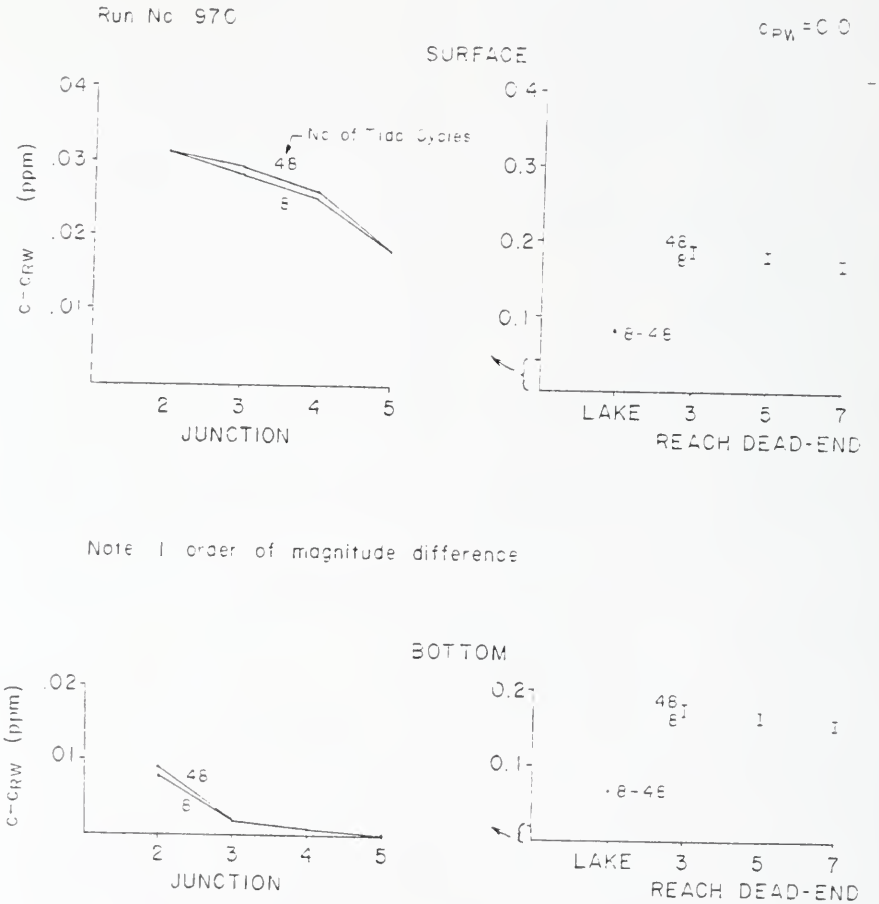


Figure 8.41 - Values of Surface and Bottom Concentrations for East Wind at Junctions and Dead-Ends at High Tide After Forty-Eight Tidal Cycles, System B.

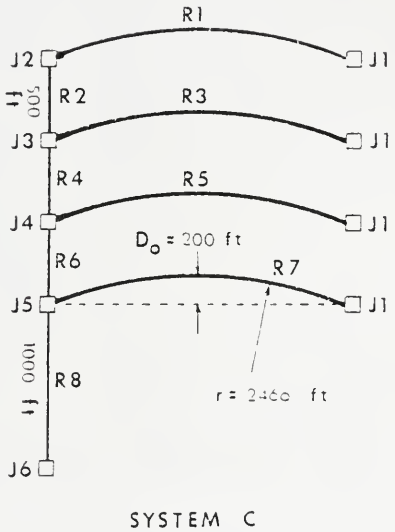


Figure 8.42 - Layout and Dimensions of System C.

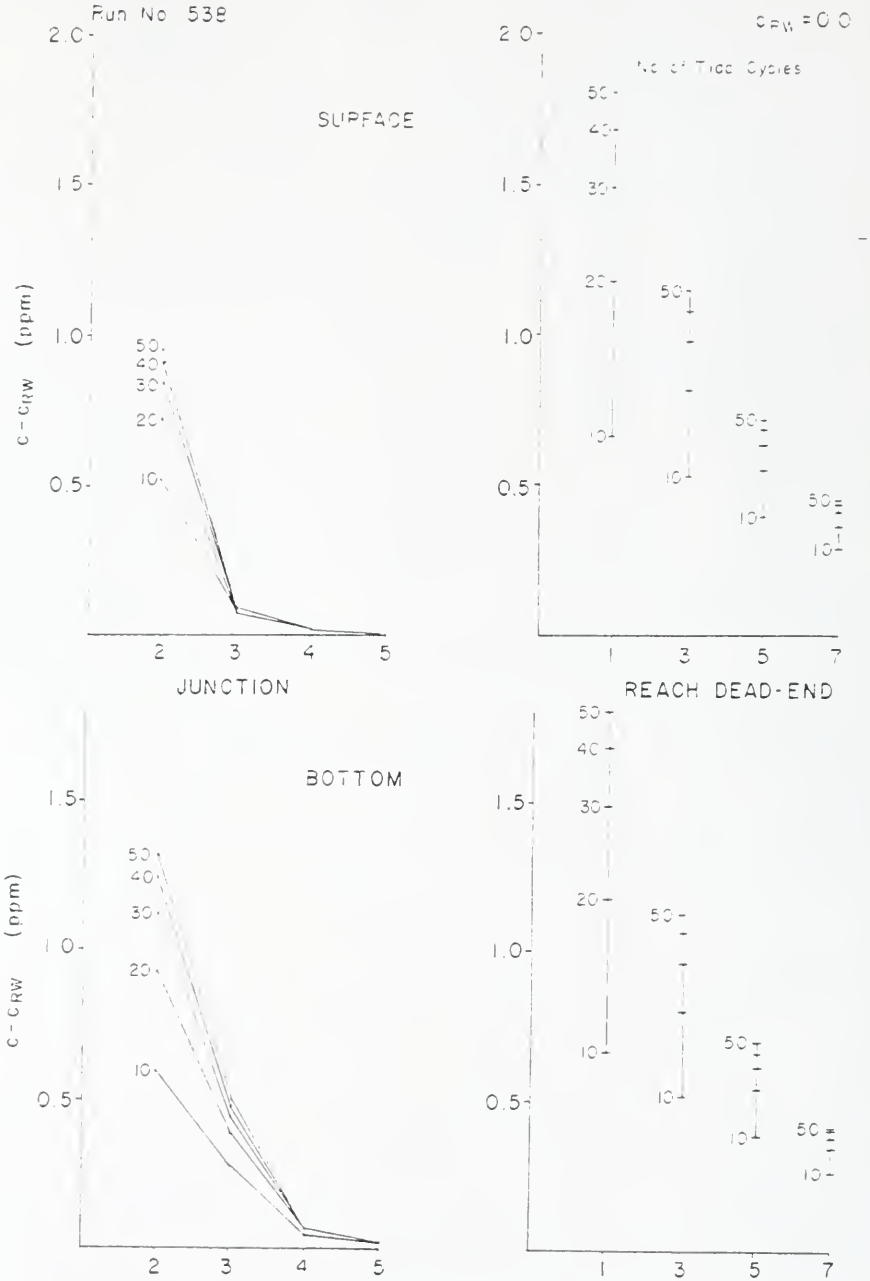


Figure 8.43 - Values of Surface and Bottom Concentrations for Upstream Wind, at Junctions and Dead-Ends at High Tide After Fifty Tidal Cycles, System C.

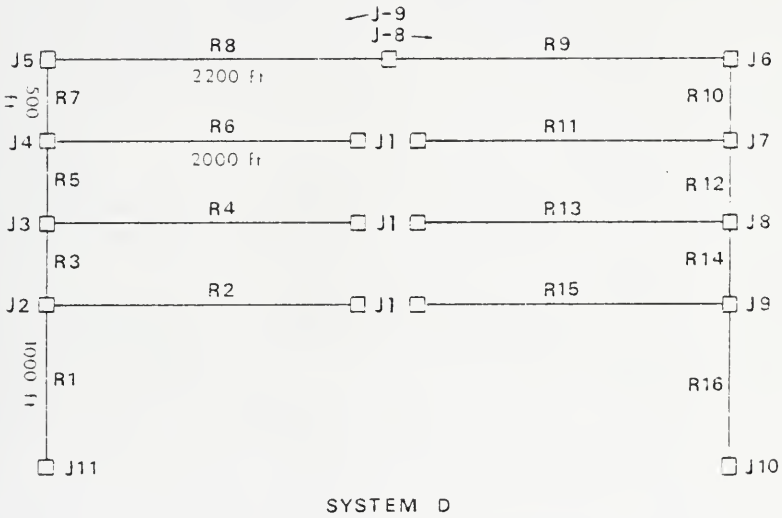


Figure 8.44 - Layout and Dimensions of System D.

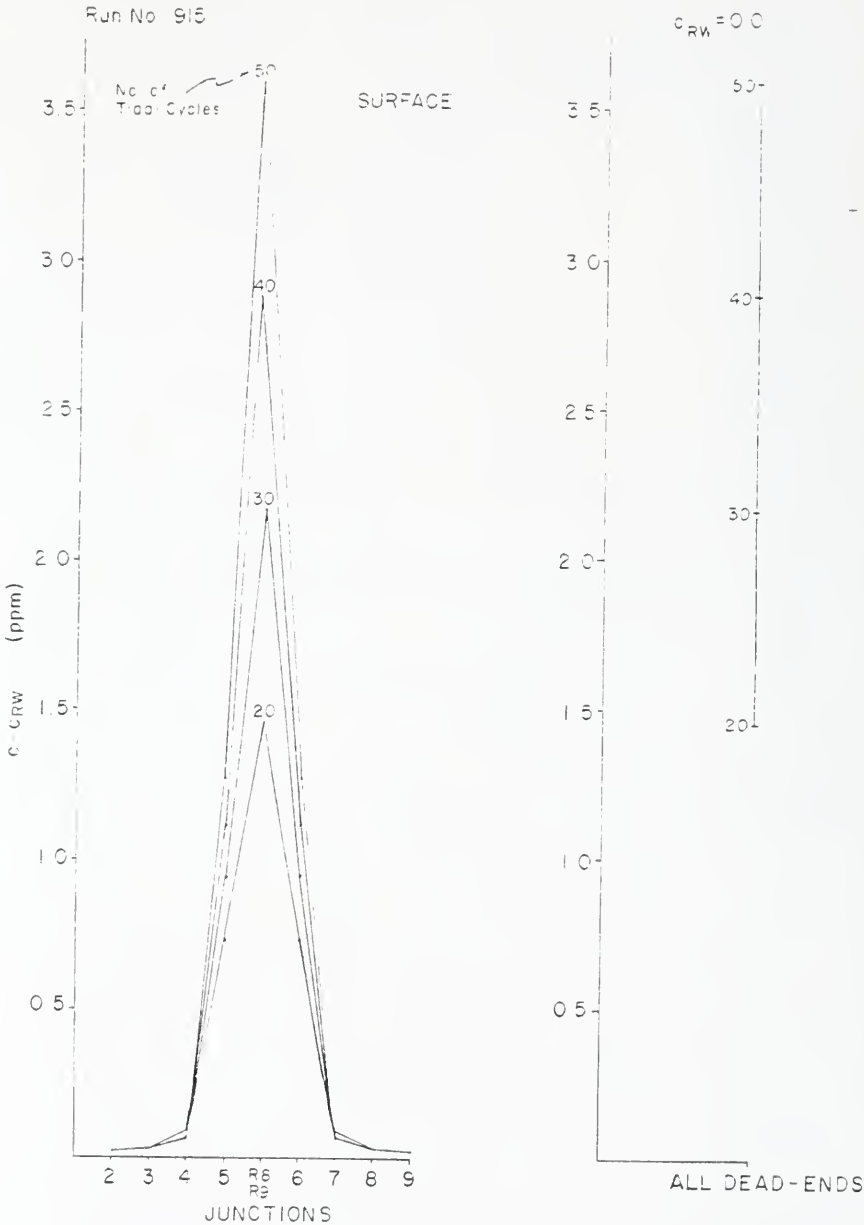


Figure 8.45a - Values of Surface Concentrations for No Wind at Junctions and Dead-Ends at High Tide After Fifty Tidal Cycles, System D.

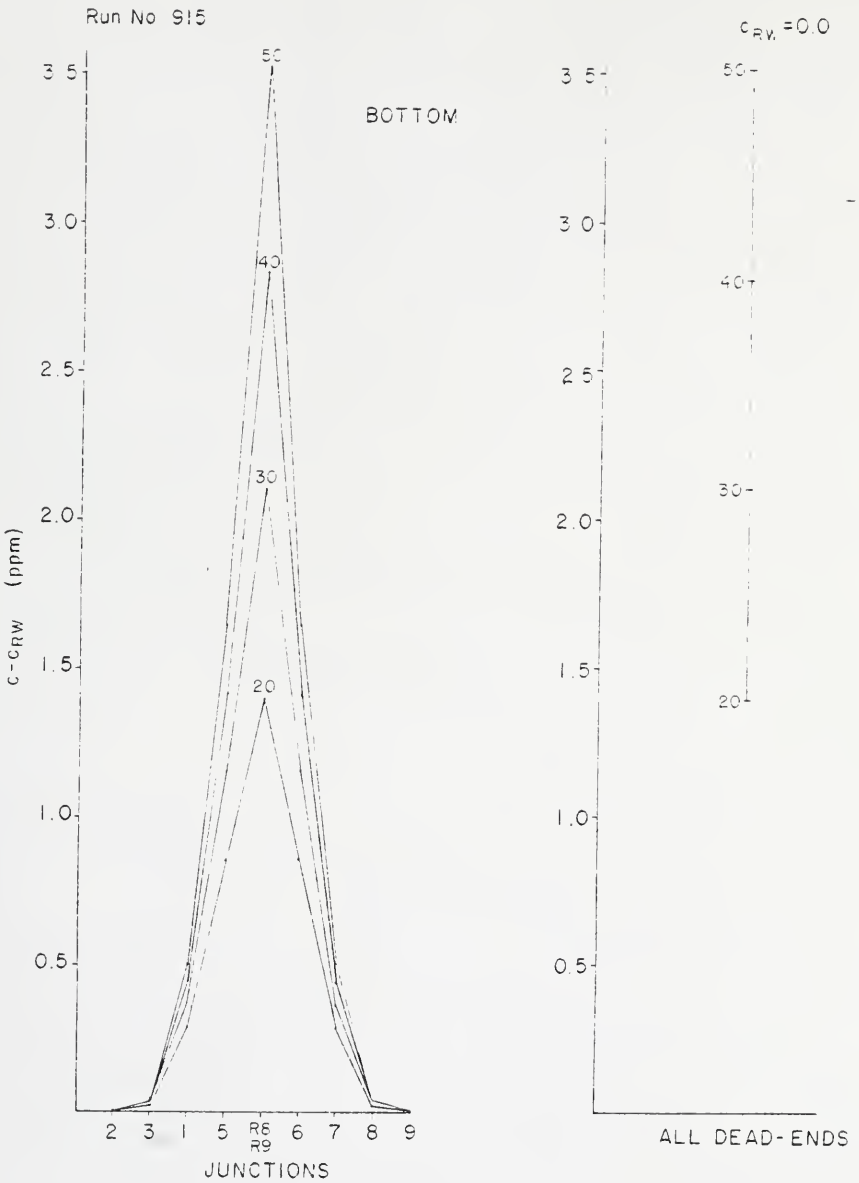


Figure 8.45b - Values of Bottom Concentrations for No Wind at Junctions and Dead-Ends at High Tide After Fifty Tidal Cycles, System D.

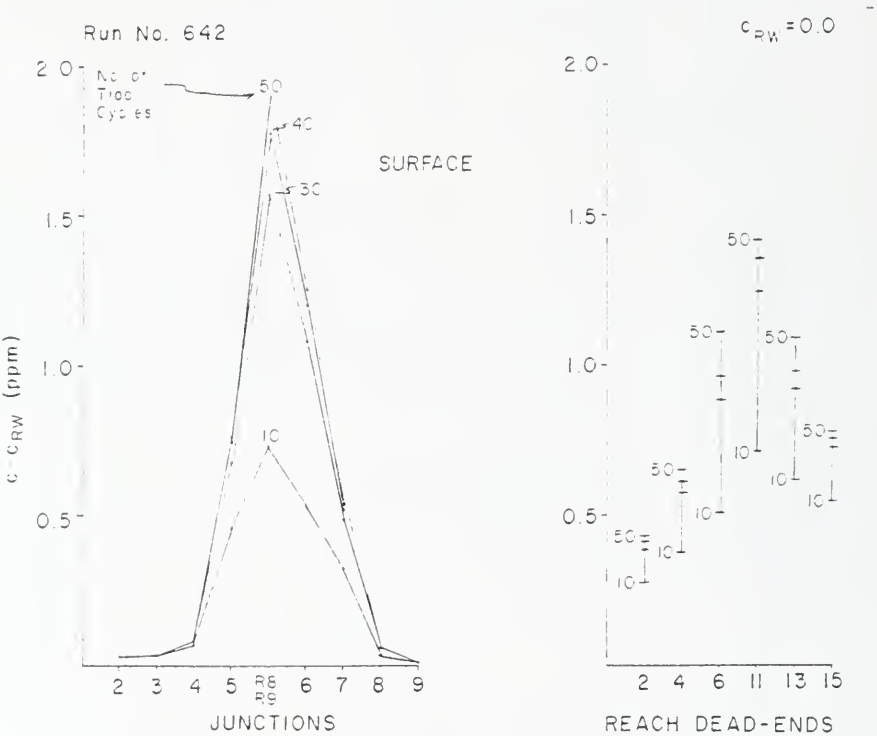


Figure 8.46a - Values of Surface Concentrations for East Wind, at Junctions and Dead-Ends at High Tide After Fifty Tidal Cycles, System D.

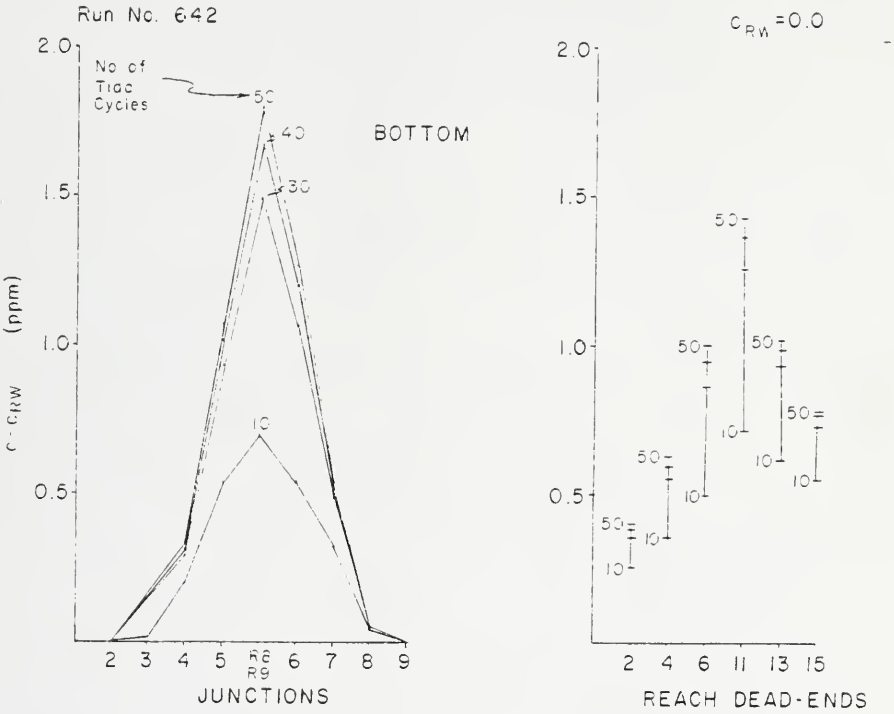


Figure 8.46b - Values of Bottom Concentrations for East Wind, at Junctions and Dead-Ends at High Tide After Fifty Tidal Cycles, System D.

CHAPTER 9

DESIGN ALTERNATIVES AND EXAMPLE OF MODIFICATION OF AN EXISTING CANAL DESIGN

9.1 Introduction to the Example

In order to illustrate a method by which an existing canal design can be iterated toward a final design that meets the established design criteria and constraints, a *simplified* network will be simulated in this chapter. The design has been synthesized from several of the comb-structured canal elements described in Section 8.5. The network has been purposely simplified to enable comparisons of concentration profiles in various portions of the network to be easily made, and the effects of changes in design to be isolated and discussed. Bends will not be included in the design to minimize costs of computation. This design example will be based on an assumption that only one tidal entrance can be used, since this is the most common arrangement for residential canals along the southern and Gulf coasts.

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

9.2 Description of Existing Canal System

Figure 9.1 is a diagram of the "existing" canal system which has been developed for this example. The site has 5000 ft of tidal shore-

line and a depth of 5000 ft to the main road, an area of 574 acres. A small pond, 6 acres in surface area, lies near the main road. Water supply and sewerage and storm water connections are offsite.

The sides of the 75-ft-wide canals 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 the collection of trash at the downwind ends of finger canals have been experienced. The owner has decided to modify the canal banks 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. The mean tidal amplitude is 1 ft and the wind is predominantly from the East during the summer.

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 variability tests in Section 8.5 (see Table 8.6) except that the length of all finger canals is 1000 ft.

9.3 The Canal Network Model

In the interests of economy with regard to computer simulation, the two-dimensional option of CANNET3D should always be used unless there is a particular need to study in three-dimensions a special feature that may have a critical three-dimensional effect on the performance of the network. 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 thereby 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 "lake" (a large junction) which 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 9.2) 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 8.6.2, 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 9.2). When the lake and its associated reach are added, they are

designated J10 and R19 respectively. The tidal entrance is labeled junction 10 without the lake, and junction 11 when the lake is included.

Results of all tests on the network are plotted on a common form to facilitate comparison (for example, see Figure 9.3). This form is arranged to scale, except 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 50 tidal cycles (621 hours, or over 25 days), and all plotted values of concentration are cross-sectionally averaged high tide values. Thus, the degree of flushing after 50 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 is available from the computer printout.

A simulation of wind-induced flushing in the existing, bulkheaded canal network is shown in Figure 9.3. This demonstrates that the system flushes relatively uniformly under this steady wind.

9.4 Simulation Objectives

Before beginning the simulation of a trial canal design, some consideration should be given to the kinds of results that will be required for making design decisions. Specifically, the conditions that need to be simulated to provide reasonable assurance that the pre-

dictions 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 establishing the values of diffusion coefficients.
2. Simulation of different conditions in the same or a similar network for validating the settings in the model.
3. Variability tests of the trial network design to determine network configuration and geometry.
4. Simulations with typical 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.

Steps 1 and 2 above have been described in Chapter 7.

The network variability tests (Step 3 above) require the introduction of a substance into the canal network that 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, and the background concentration to a value of 1. 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. 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 condition, which will flush considerably faster, is a high initial point source concentration at one or more locations in the network. This procedure, however, is good only for observing the flushing rate at one point, or perhaps several widely separated points, in the network.

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.

9.5 Flushing Under No-Wind Conditions

Under no-wind conditions the trial network flushes very poorly. Figure 9.4 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 located to the tidal entrance.

When the mean depth of the canals is halved to a value of 4.0, flushing is slightly, but not significantly, increased (Figure 9.5). In a 4-ft-deep canal the tidal influence reaches to the ends of the finger canals in the western part of the network, and most of the way up the main canal in the eastern part, since the tidal prism occupies a greater portion of the total volume of the system. However, flushing is again relatively poor without wind in the upstream halves of the finger canals in the western portion, and not at all effective in any part of the double set of finger canals.

9.6 Design Alternatives

9.6.1 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 9.6). 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 9.7). In the case with a 2 mph wind, flushing is still predominantly due to tidal action, while the higher wind has completely changed the concentration

pattern to a comparatively uniform distribution.

Compared with the results for the vertically bulkheaded original design (Figure 9.3), 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.

A similar comparison has been made with a 4-ft-deep canal. When a steady wind of 5 mph is superimposed on this network the concentration profiles are again substantially decreased, although not as much as in the 8-ft-deep canals (Figure 9.8). The concentration profiles are relatively flat, and it can be seen that no portions of the canal network have any difficulty being flushed.

The characteristics of a 12-ft-deep canal cannot be extrapolated from the comparison between the 8- and 4-ft canals. The results for a 5 mph wind in a 12-ft canal (Figure 9.9) show the poorest flushing of the three cases in the western portion, and intermediate flushing in the eastern portion. Thus, it can be tentatively concluded that, at least for the particular network configuration and wind speed and direction tested above, there exists an optimal depth in the neighborhood of 8 ft.

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 9.10, 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

$$\frac{c}{c_o} = K_o e^{Kt} \tag{9.1}$$

where

c_o = background concentration, (dimensionless)

K_o = constant associated with an intitial value, (dimensionless)

e = the exponential constant

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 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 and N represents the number of tidal cycles since the start of the simulation,

$$K = - \frac{\ln(c_1/c_2)}{N_2 - N_1} \tag{9.2}$$

and

$$N_{10\%} = N_1 - \frac{\ln c_1}{K} \tag{9.3}$$

where

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

Calculated flushing times for the simulations with winds are summarized in Table 9.1.

Figure 9.10 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 30 through 50 tidal cycles lie on a straight

line on the semi-logarithmic coordinates. Thus, if the assumption that the decrease in bottom concentration follows a first-order decay relationship is valid, these lines may be easily extrapolated to 10 percent ($c/c_o = 1$) by using Equation (9.2) and (9.3). The resulting flushing times are 92 tidal cycles for the dead-end of canal R2 and 136 tidal cycles for the dead-end of canal R12. Similarly, the extrapolated flushing times for the other cases with wind, and one without, are summarized in Figures 9.11 and 9.12 and in Table 9.1.

If the effects of a range of mean depths and steady winds are considered in the context of the results of the simulations described thus far, it is evident (from Table 9.1) that the 8 ft depth is the best choice for the canal network, of the three depths tested. If prevailing winds were much less than 2 mph, the 4 ft depth would be better for maximizing tidal flushing on the average than the 8 ft depth, but it is not likely that winds would be limited to such a low value over an extended period of time in Florida. This comparison does illustrate, however, how limited the flushing can be in an extensive canal network during periods when the wind is very light and there are no intervals of wind strong enough to induce even temporary mixing.

9.6.2 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 results in 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 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 will have some effect on the downstream portions of the finger canals.

The surface area of the trial canal network is 1,680,000 ft^2 (38.6 acre). The surface areas of the lake, which have been included in simulations of both 4- and 8-ft-deep networks for purposes of evaluating the effect of additional tidal prism on the flushing of the network, are 250,000 ft^2 (5.7 acre, 15 percent of total network area) and 500,000 ft^2 (11.5 acre). The results for the 8-ft-deep network, no wind, and the smaller lake area, are shown in Figure 9.13. The results of this simulation show that the concentration values in the main channels are reduced approximately 50 percent after 50 tidal cycles, and that the increased flushing extends, with decreasing effect, almost the entire distance into the finger canals. Comparison with Figure 9.4 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 9.14), 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 50 tidal cycles, with proportional 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. Simulations with the 4-ft-deep alternate design and the 30 percent lake area result in almost exactly the same results in the main channels (Figure 9.15). Reductions of more than 10 percent at the dead-ends of 4-ft-deep

finger canals are a slight improvement over comparative results in the 8-ft-deep network.

When combined with an east wind of 5 mph in the 8-ft-deep system, however, the concentration profiles in the eastern portion of the network are uniformly reduced to 12 percent of their initial vaues after 50 tidal cycles (Figure 9.16). Comparing Figure 9.16 with the results from identical conditions, except for no lake, (Figure 9.7) 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. A simulation with the 4-ft-deep alternate design, an east wind of 5 mph, and a lake area of 30 percent produced uniform concentration profiles of 10 percent or less throughout the eastern portion of the network (canals R1 through R10) in less than 30 tidal cycles (Figure 9.17).

The cases with lakes are compared by means of flushing curves for bottom concentrations in canal R2 in Figure 9.18. The two cases for the 8-ft-deep network with no wind and both 15 percent and 30 percent lake surface area, do not flush well enough to warrant calculation of a flushing time. The 4-ft-deep canal with no wind and 30 percent lake surface area has a calculated flushing rate of 500 tidal cycles to 10 percent of initial concentration, exactly the same as calculated for the case with no lake (run no. 553, Table 9.1). The two cases with winds, one an 8-ft-deep canal (run no. 524) and the other the 4-ft-deep canal (run no. 663) flush extremely rapidly and not logarithmically. The combination of wind and extra tidal prism is, from the simulations presented in this section, obviously the most effective flushing mechanism presented thus far for tidal canal networks.

9.6.3 Comparison of Effects of Steady and Variable Wind

From the comparative tests described above, it is evident that the results of a given simulation depend more upon the wind than upon the depth or the tide. For these 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 seem to be appropriate to use a typical time-varying wind and tide to ensure more realistic results. However, if typical time-varying data are not available for two or more different seasons at the site, the question might be asked: can an "equivalent" steady wind be found which will produce results equivalent to those which would be obtained using field data?

To explore this question, a variable wind sequence was used with the harmonic tides to obtain a network concentration profile after 50 tidal cycles. This wind sequence was defined as an approximation to the 57 Acres measured wind data by specifying a typical wind speed and direction at four times during the day and using the model, CANNET3D, 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 night is typical for 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 the 57 Acres site, 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 with the 0300 wind.

An "equivalent" steady wind was defined next by noting that the shape of the wind velocity distribution is approximately Gaussian over the 18-hr period during which the variable wind is active. Taking the peak value to be 8 mph, numerical integration at 1-hr 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 simulating a 2 mph wind from 090 were shown in Figure 9.6. The results from a simulation using the "equivalent" variable wind are shown in Figure 9.19. 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 have a very limited effect.

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. It was mentioned in Section 12.1.3, from one comparison, that 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, published data are not transferable to different sites, and a steady "equivalent" wind cannot be calculated from a variable wind distribution, it follows that the canal designer must 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. The simulations conducted for the evaluation of the hypothetical rectangular canal and the proposed trapezodial canal are summarized in Table 9.2.

Table 9.1 - Relative Calculated Times to Reduce Bottom Concentration at the Two Slowest-Flushing Dead-Ends to 10 Percent of Their Initial Value, Arranged in Order of Decreasing Flushing Time, for the Example Canal Design.

Dead-end of Canal No.	Mean Depth (ft)	East Wind Speed (mph)	Flushing Time to 10 Percent		Run No.
			Tidal Cycles	Days	
R2	8	2	9059	4688	32
	4	None	500	259	553
	4	5	292	151	182
	12	5	181	94	728
	8	5	92	47	444
R12	4	None	1161	601	553
	12	5	943	488	728
	8	2	415	215	32
	4	5	218	113	182
	8	5	136	70	444

Table 9.2 - Summary of Simulations of the Original, the Trial and Some Alternative Canal Network Designs.

Design	Mean Depth (ft)	mph/deg.	Lake Area (ft^2)	Flushing Time to 10% less than 1000 tidal cycles for vertically averaged concentrations at dead-ends of reaches: R1	R12	Run No.	Figure No.
Original	8.0	5/090	none	no	no	827	9.3
Trial	8.0	none	none	no	no	443	9.4
	8.0	2/090	none	no	yes	32	9.6
	8.0	5/090	none	yes	yes	444	9.7
	8.0	none	250,000	no	no	726	9.13
	8.0	none	500,000	no	no	451	9.14
	8.0	5/090	500,000	yes	yes	524	9.16
	8.0	variable	none	no	no	308	9.19
Alternate	4.0	none	none	no	no	486	9.5
	4.0	5/090	none	yes	yes	182	9.8
	12.0	none	none	yes	yes	728	9.9
	4.0	none	500,000	yes	no	553	9.15
	4.0	5/090	500,000	yes	yes	633	9.17

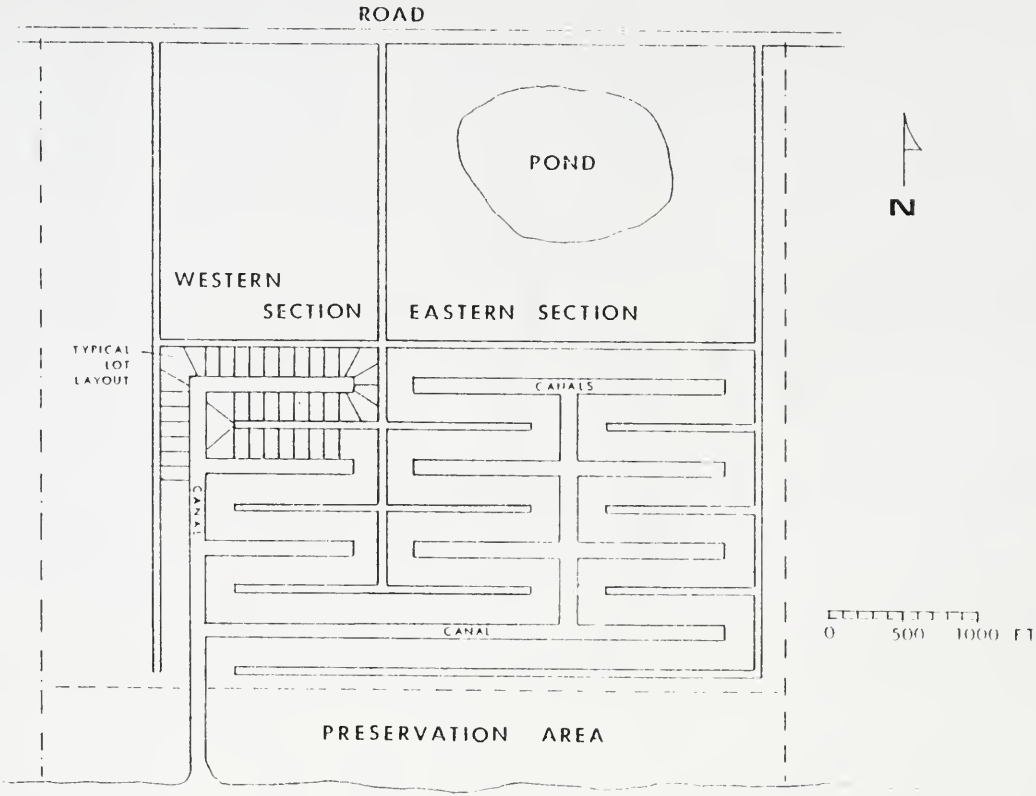
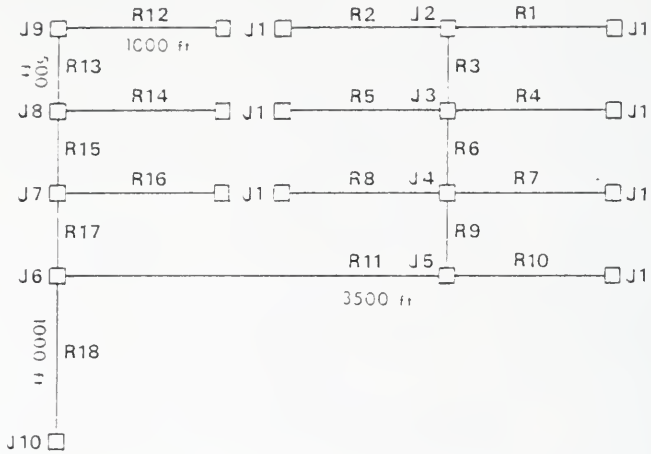
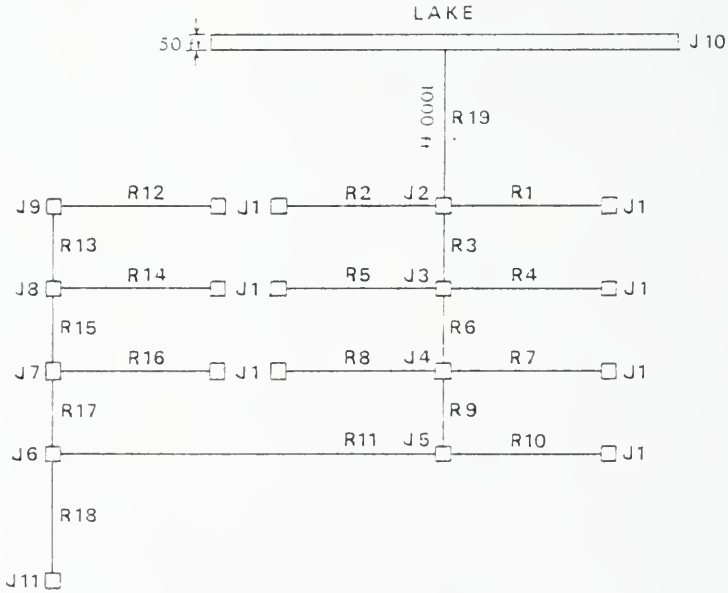


Figure 9.1 - "Existing" Example Canal System.



SYSTEM E

Note junctions are not to scale



SYSTEM EL

Figure 9.2 - Layout of Model Network

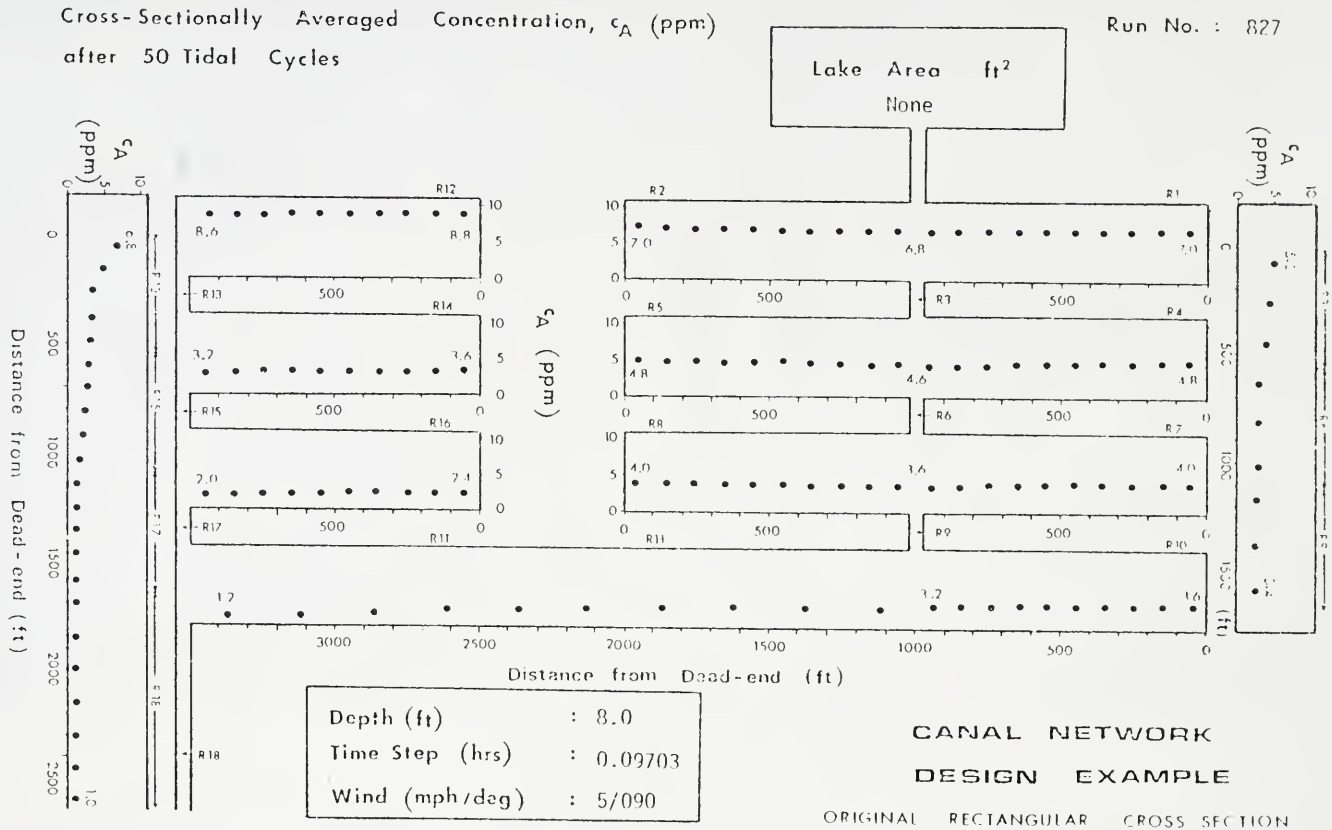


Figure 9.3 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Original Canal Network After Fifty Tidal Cycles, With a Steady Wind of 5 mph from the East (Run No. 827).

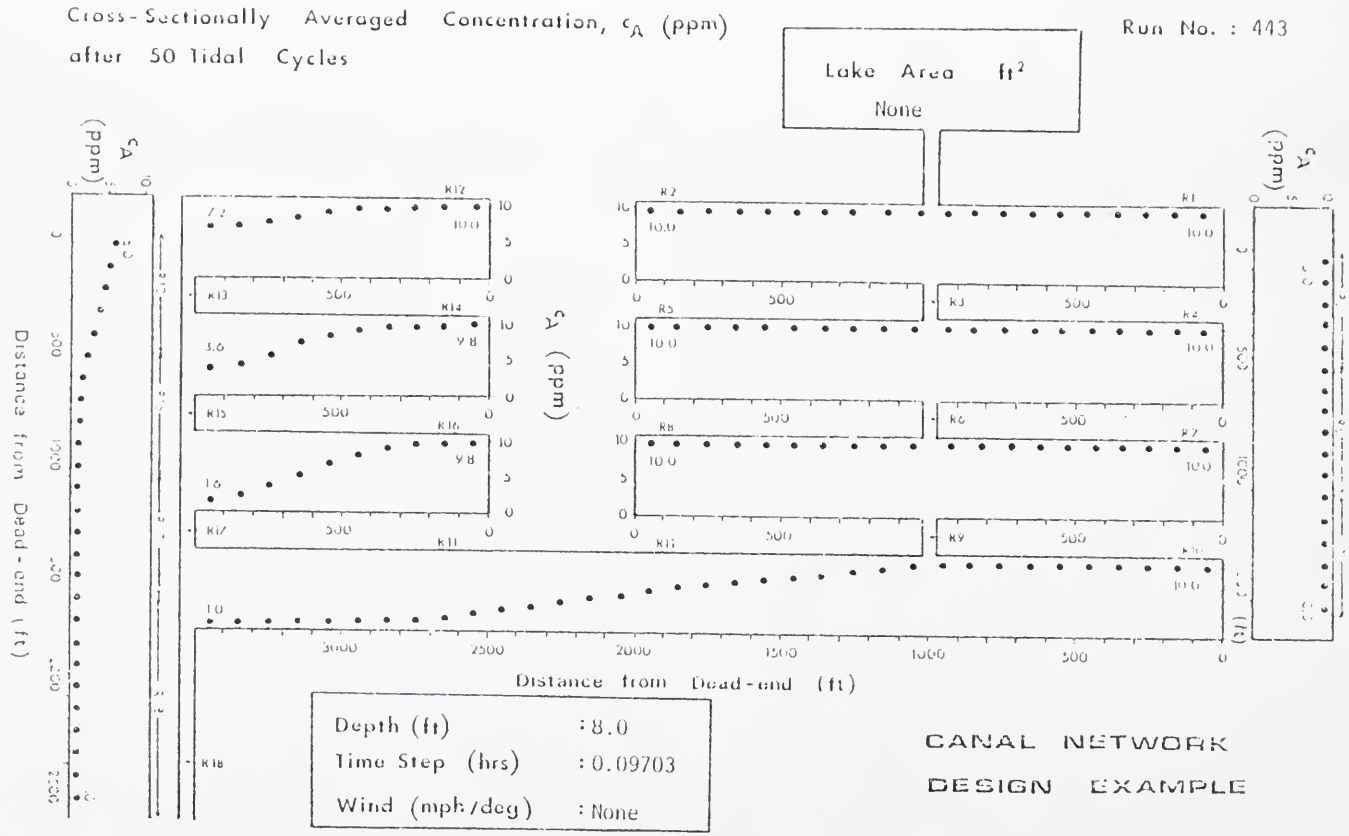


Figure 9.4 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, for No Wind and No Lake (Results of Run No. 443).

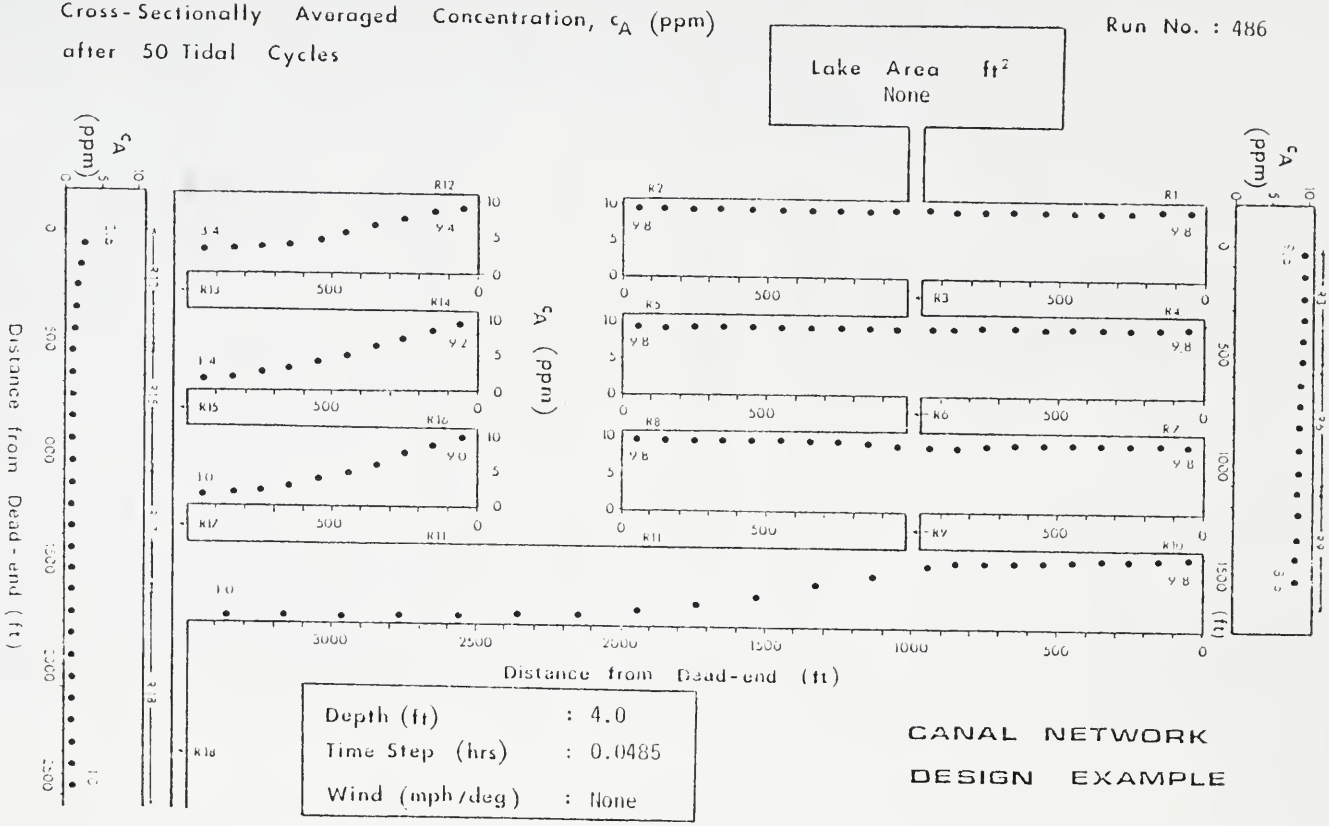


Figure 9.5 - Cross-sectionally Averaged Concentration Profiles in the 4-ft-deep Alternate Canal Network After Fifty Tidal Cycles, for No Wind and No Lake (Results of Run No. 486).

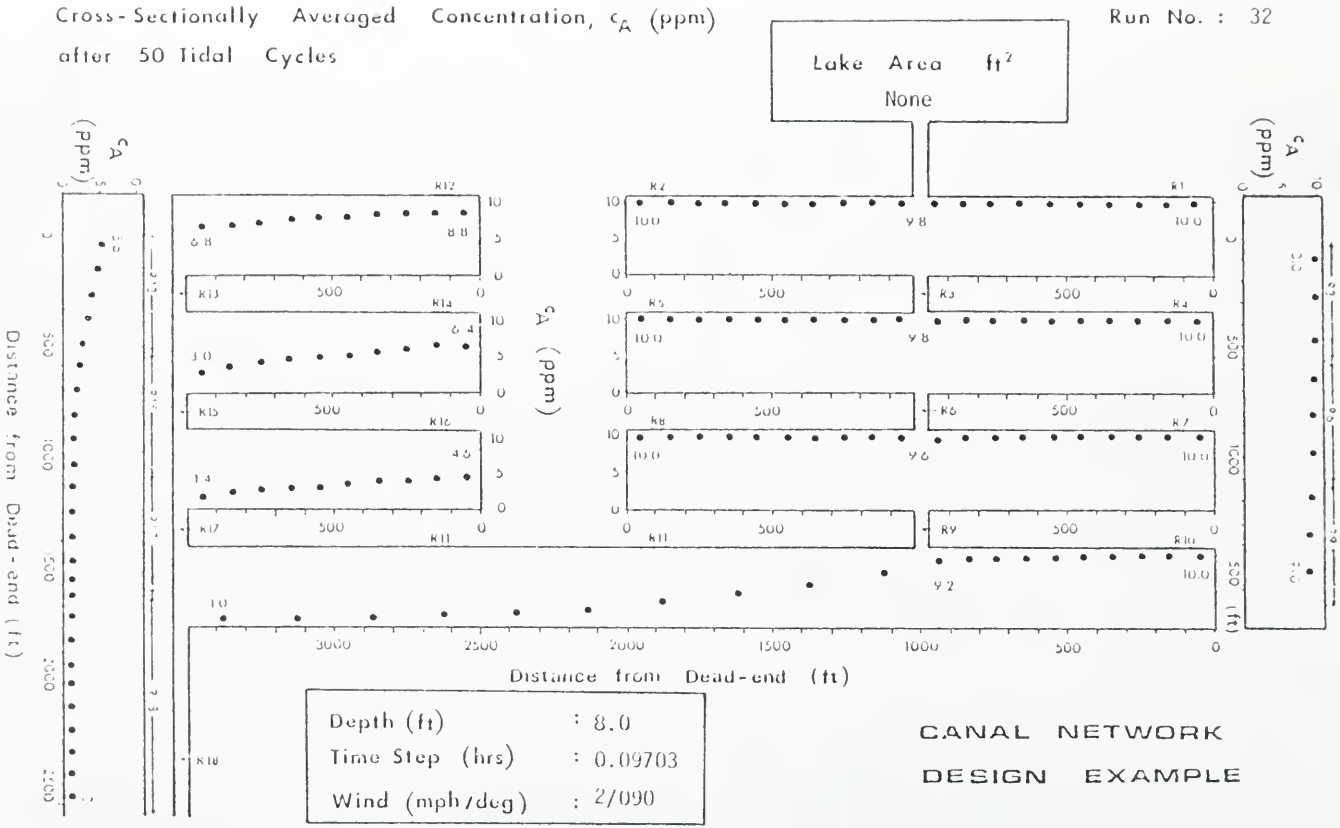


Figure 9.6 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With a Steady Wind of 2 mph from the East and No Lake (Results of Run No. 32).

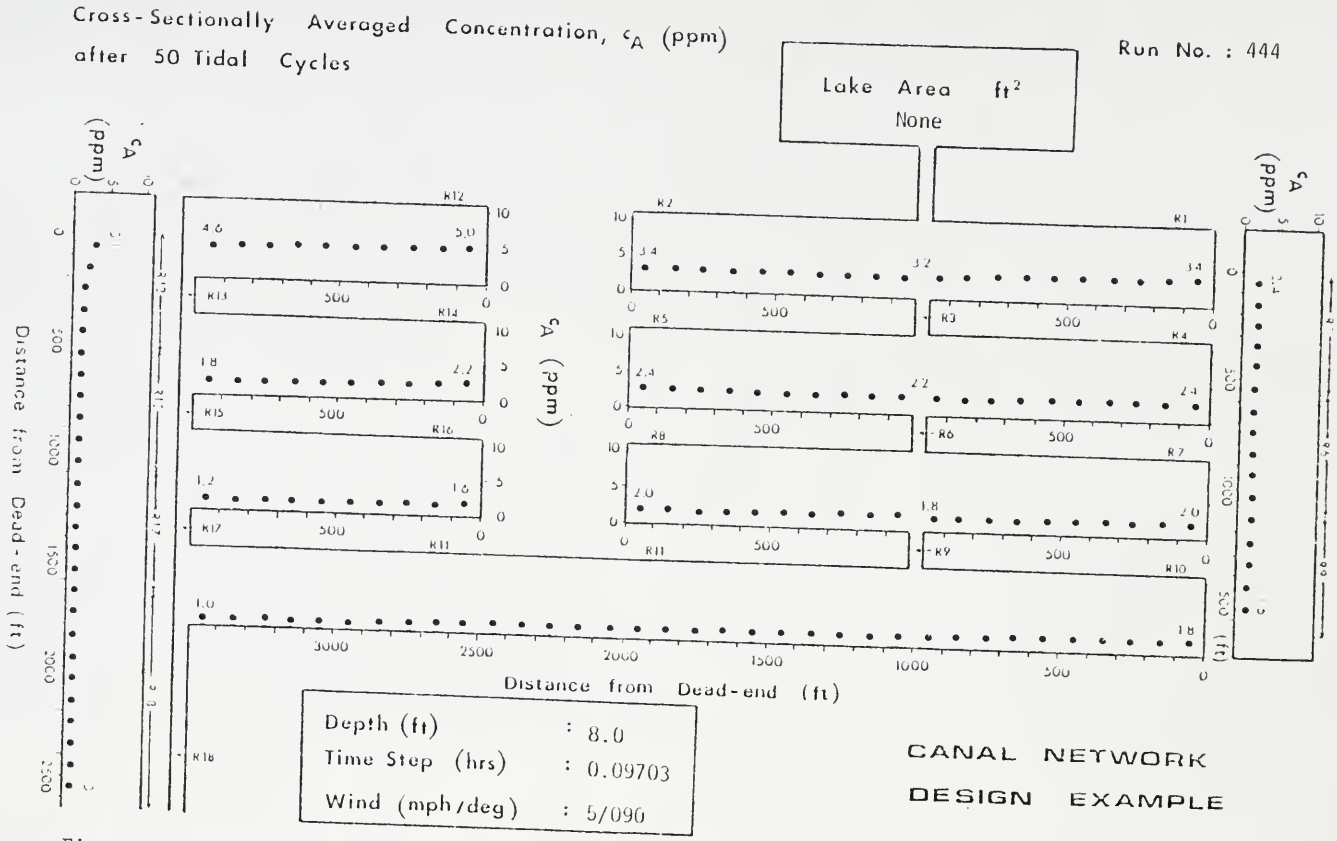


Figure 9.7 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With a Steady Wind of 5 mph from the East and No Lake (Results of Run No. 444).

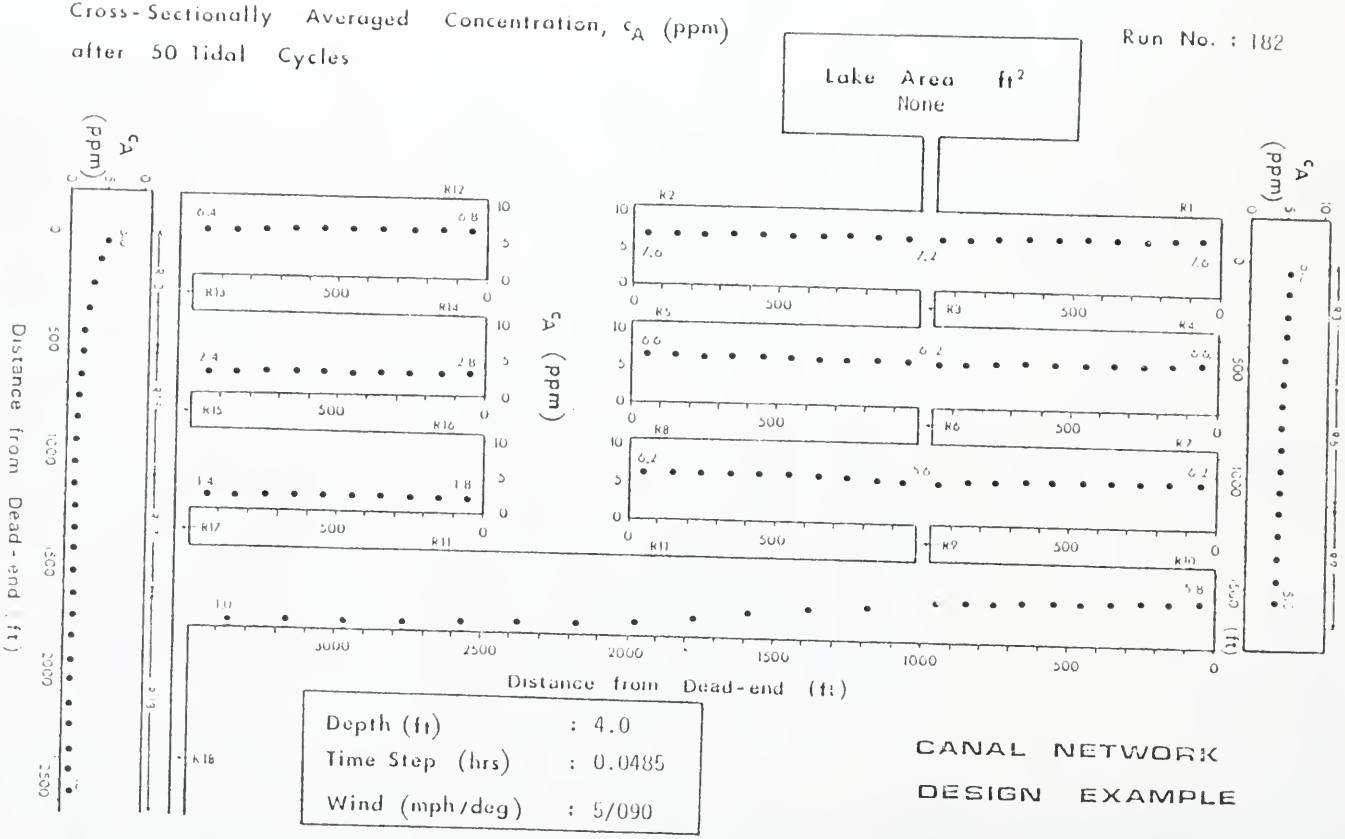


Figure 9.8 - Cross-sectionally Averaged Concentration Profiles in the 4-ft-deep Alternate Canal Network After Fifty Tidal Cycles, With a Steady Wind of 5 mph from the East and No Lake (Results of Run No. 182).

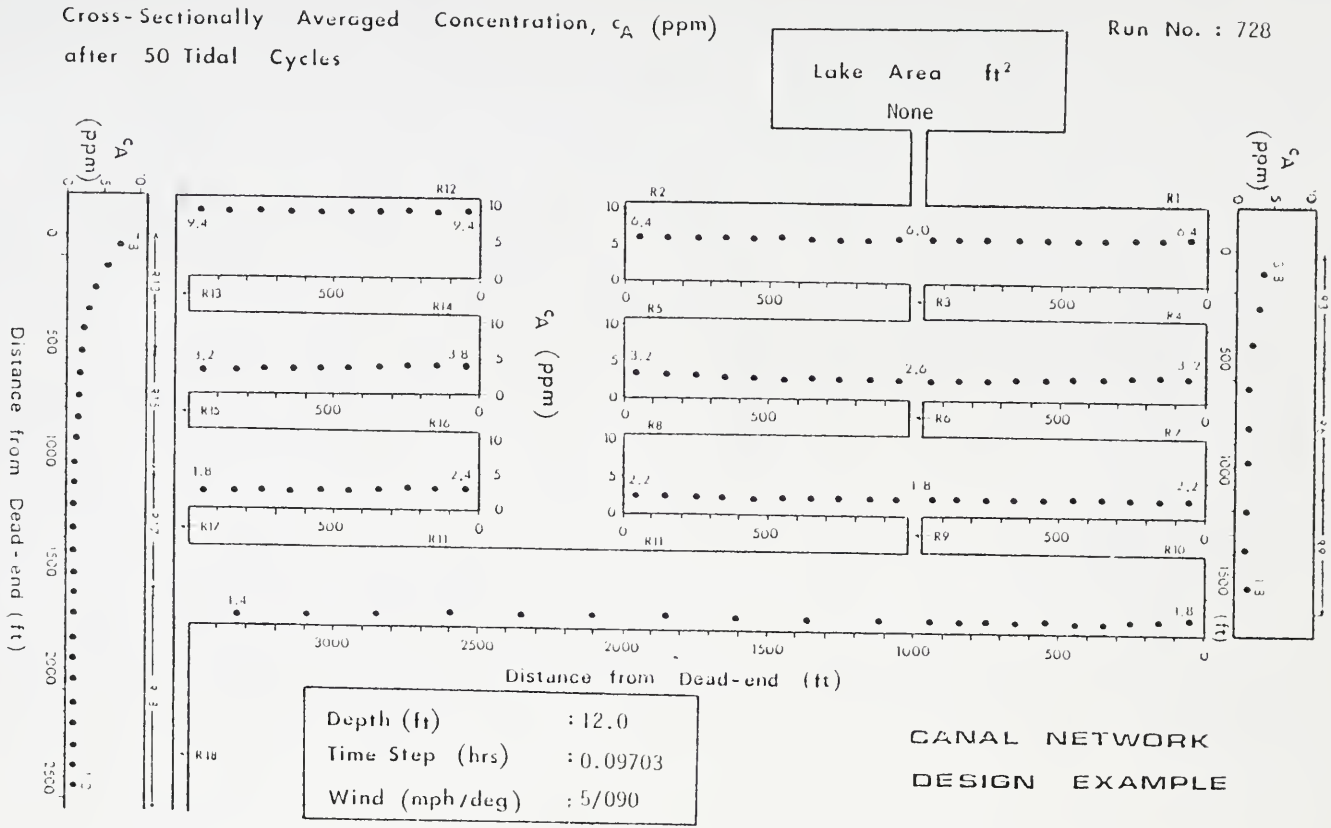


Figure 9.9 - Cross-sectionally Averaged Concentration Profiles in the 12-ft-deep Alternate Canal Network After Fifty Tidal Cycles, With a Steady Wind of 5 mph from the East and No Lake (Run No. 728).

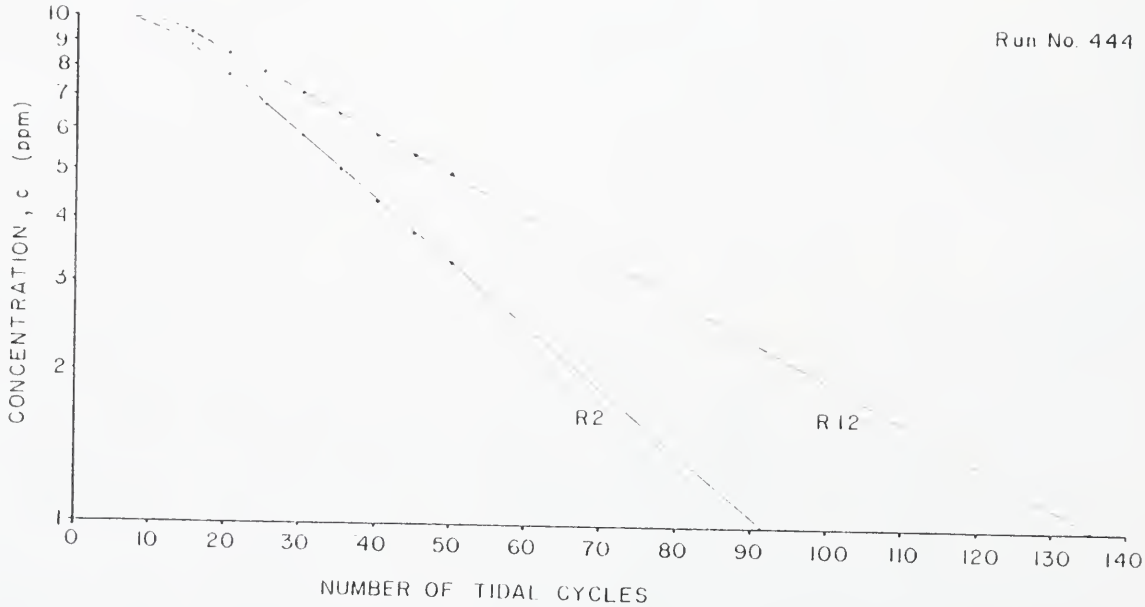


Figure 9.10 - Semilogarithmic Plots of Laterally Averaged Bottom Concentration at the Dead-ends of Canals R2 and R12 versus Number of Tidal Cycles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With a Steady Wind of 5 mph from the East and No Lake (Run No. 444).

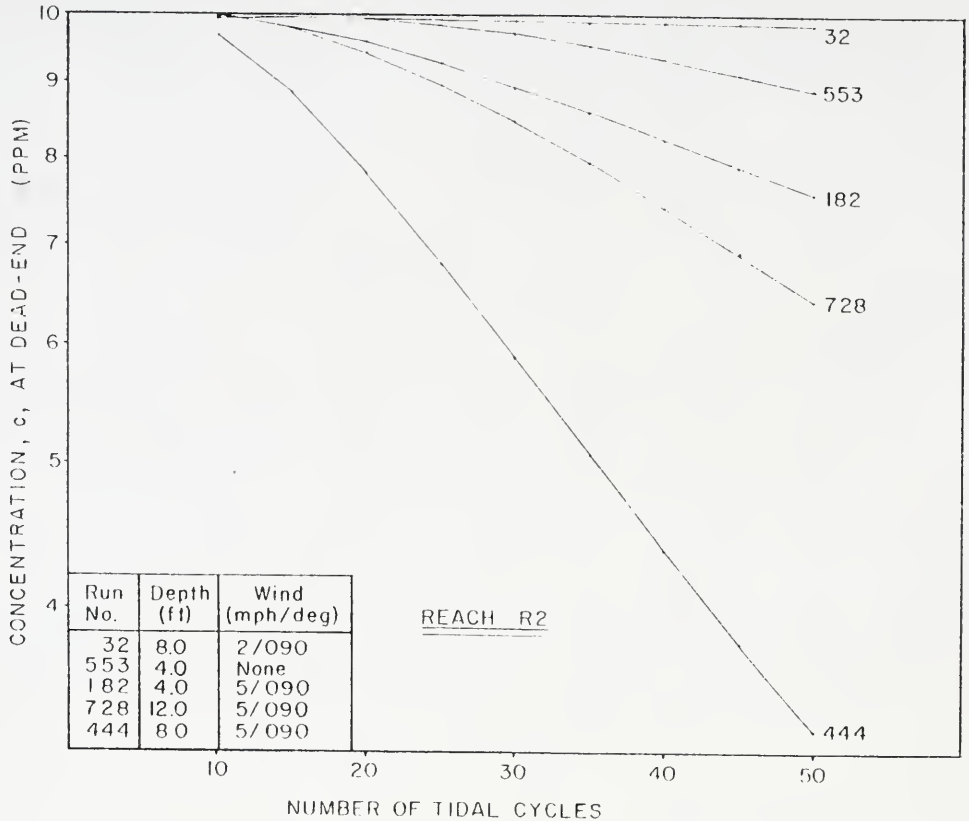


Figure 9.11 - Semilogarithmic Plots of Laterally-averaged Bottom Concentration at the Dead-ends of R2 Canals versus Number of Tidal Cycles for Various Combinations of Depths 4, 8 and 12 ft and East Winds with Velocities of 0, 2 and 5 mph (Results of Run Nos. 32, 182, 444, 553 and 728).

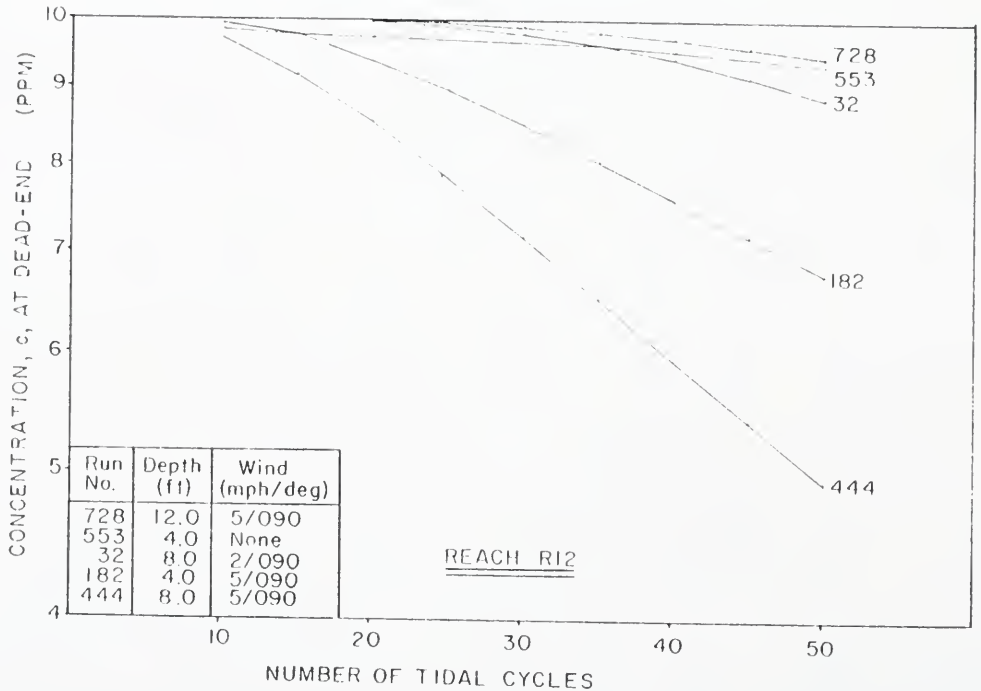


Figure 9.12 - Semilogarithmic Plots of Laterally-averaged Bottom Concentration at the Dead-ends of R12 Canals versus Number of Tidal Cycles for Various Combinations of Depths 4, 8 and 12 ft and East Winds with Velocities of 0, 2 and 5 mph (Results of Run Nos. 32, 182, 444, 553 and 728).

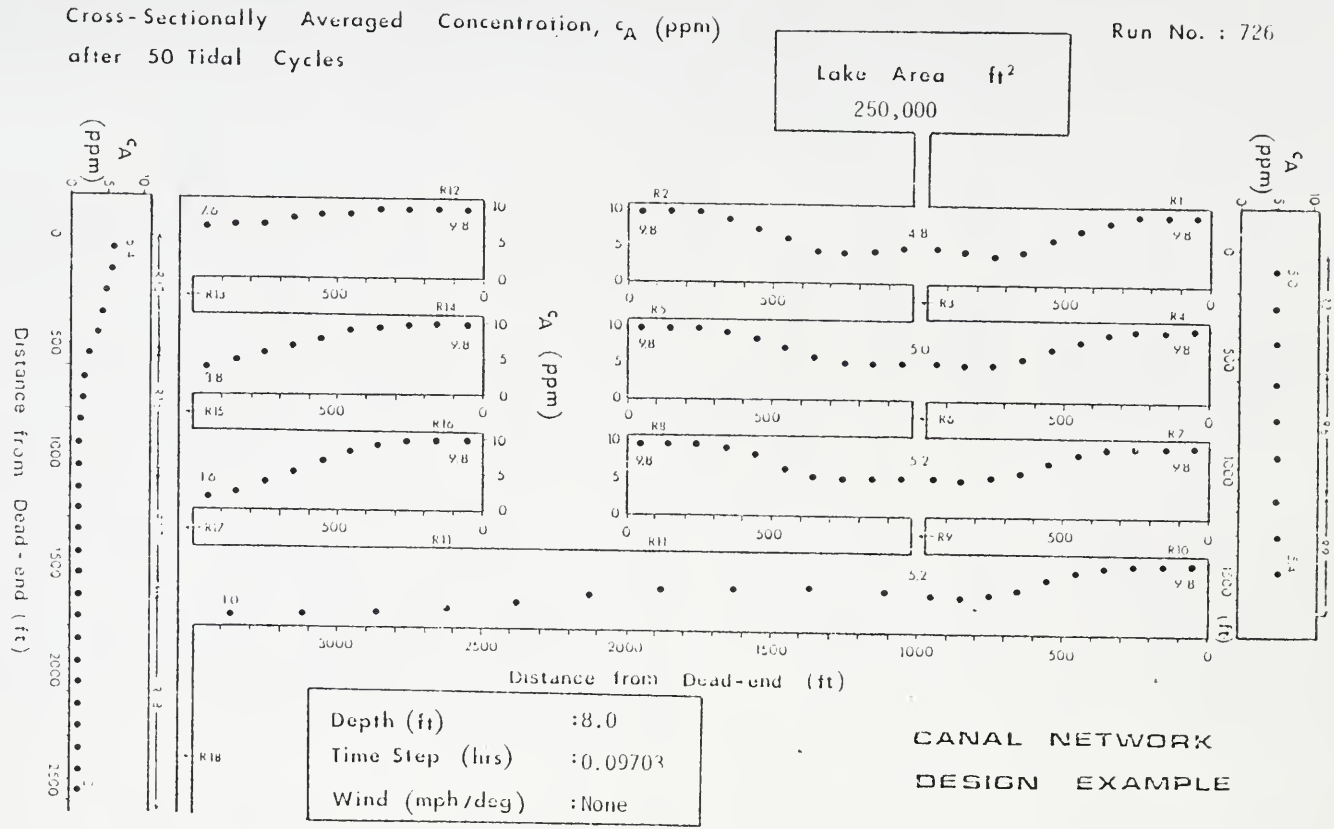


Figure 9.13 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With No Wind and a Lake With a Surface Area of 15 percent of Canal Network Surface Area (Run No. 726).

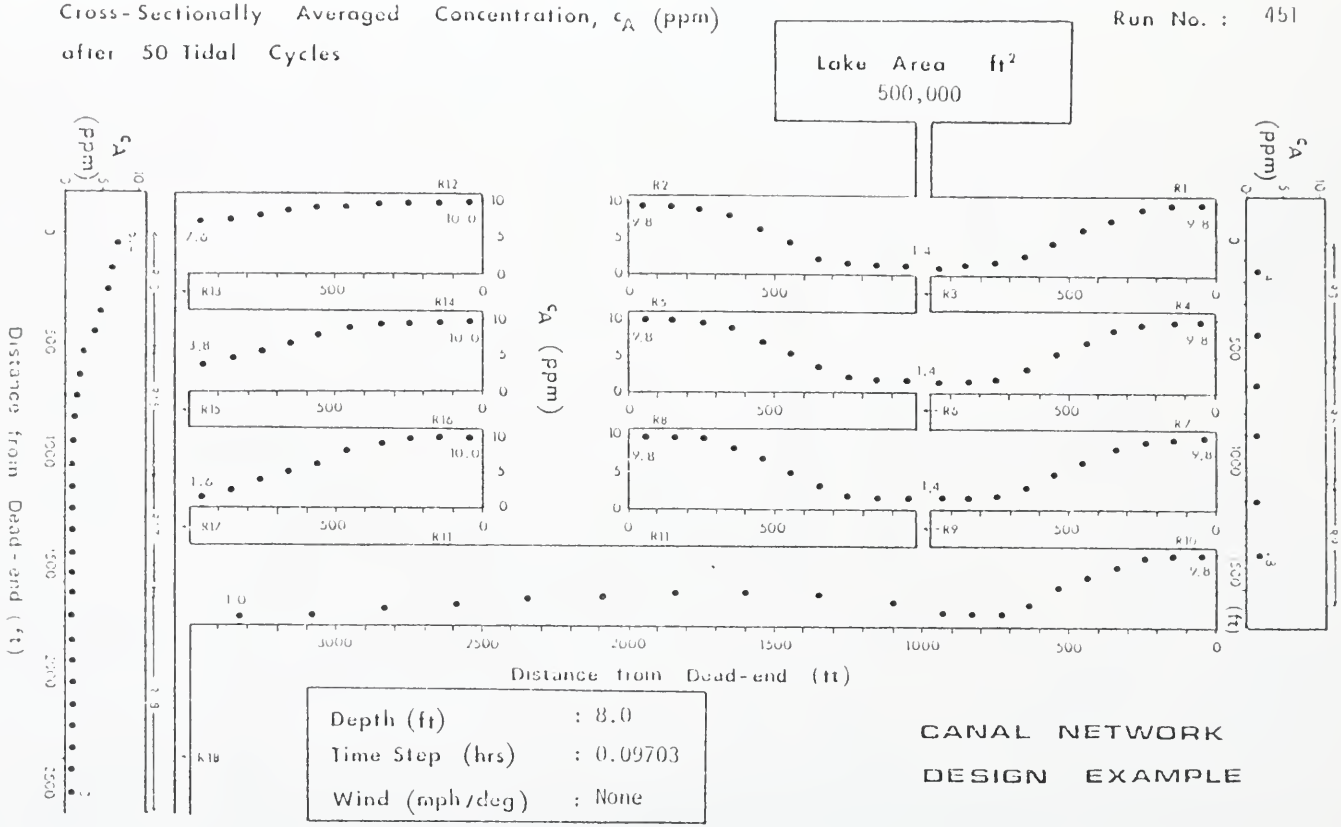


Figure 9.14 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With No Wind and a Lake With a Surface Area of 30 percent of Canal Network Surface Area (Run No. 451).

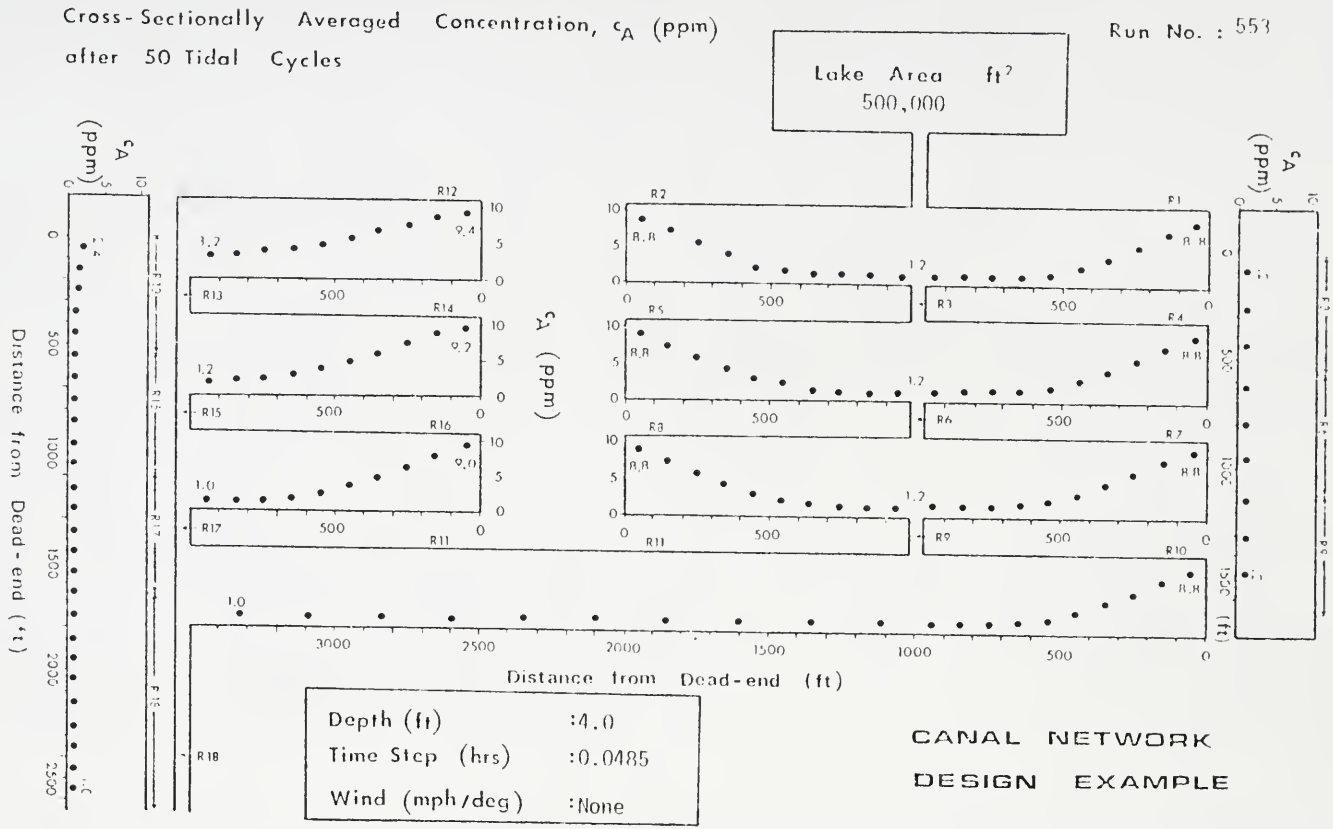


Figure 9.15 - Cross-sectionally Averaged Concentration Profiles in the 4-ft-deep Alternate Canal Network After Fifty Tidal Cycles, With No Wind and a Lake With a Surface Area of 30 percent of Canal Network Surface Area (Run No. 553).

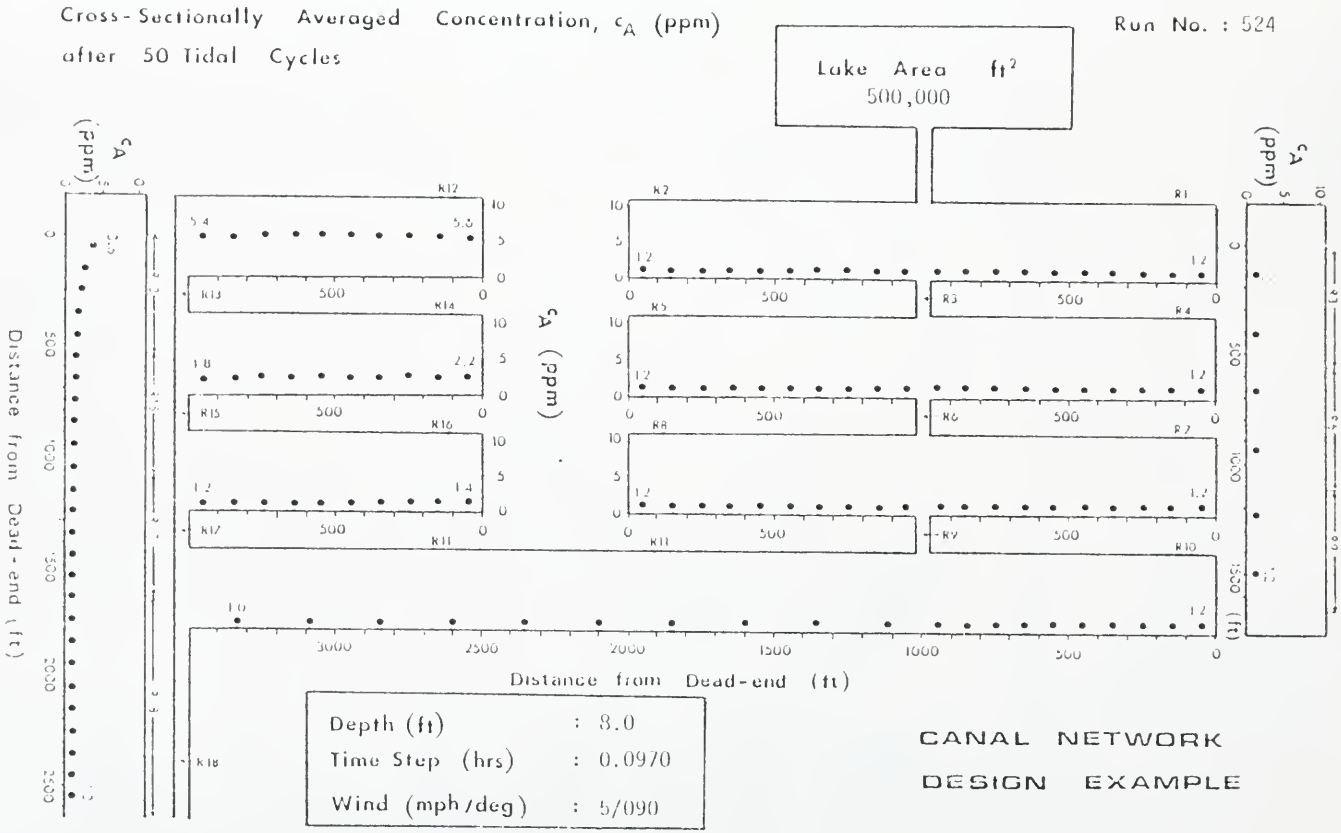


Figure 9.16 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With East Wind of 5 mph and a Lake With Surface Area of 30 percent of Canal Network Surface Area (Run No. 524).

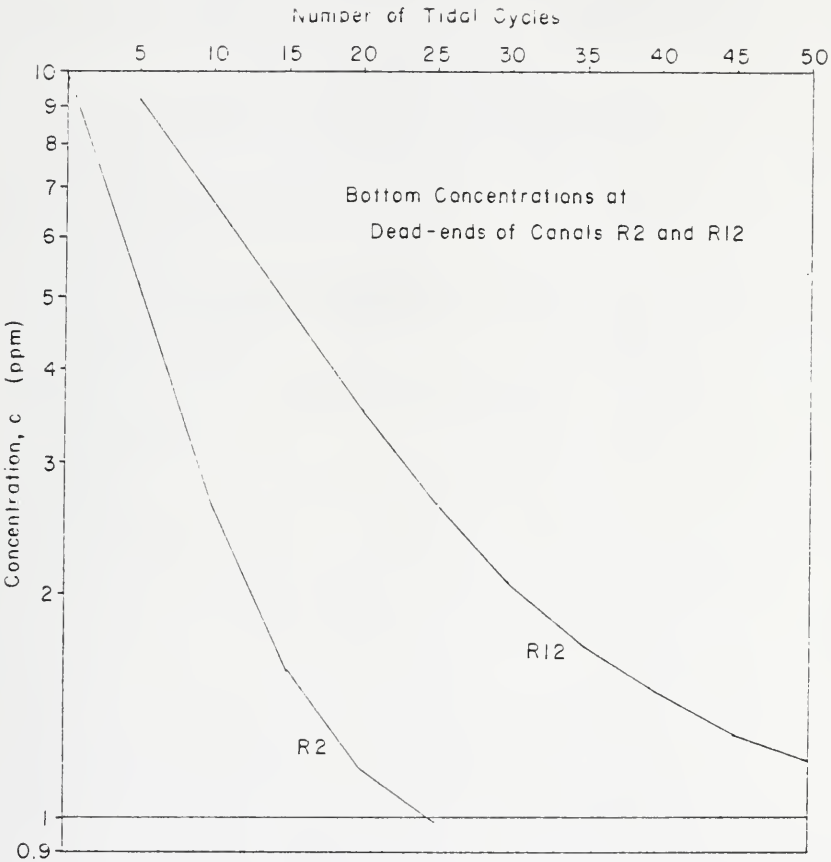


Figure 9.17 - Semilogarithmic Plot of Laterally-averaged Bottom Concentration at the Dead-ends of Canals R2 and R12 Versus Number of Tidal Cycles for 4-ft-deep Alternate Canal Network After Thirty and Fifty Tidal Cycles, With Lake Surface Area of 30 percent of Network Surface Area and With a Steady Wind of 5 mph from the East (Run No. 633).

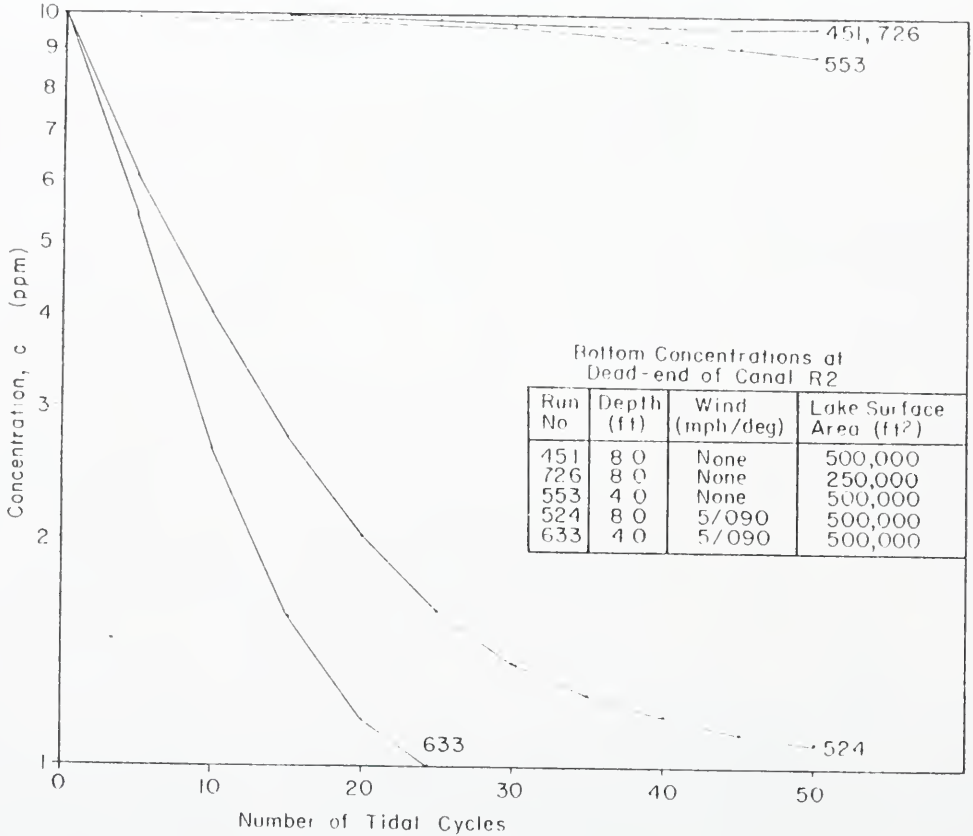


Figure 9.18 - Semilogarithmic Plots of Laterally-averaged Bottom Concentration at the Dead-ends of R2 Canals versus Number of Tidal Cycles for Various Combinations of Depths 4 and 8 ft and East Winds with a Velocity of 5 mph (Results of Run Nos. 451, 524, 553, 633 and 726).

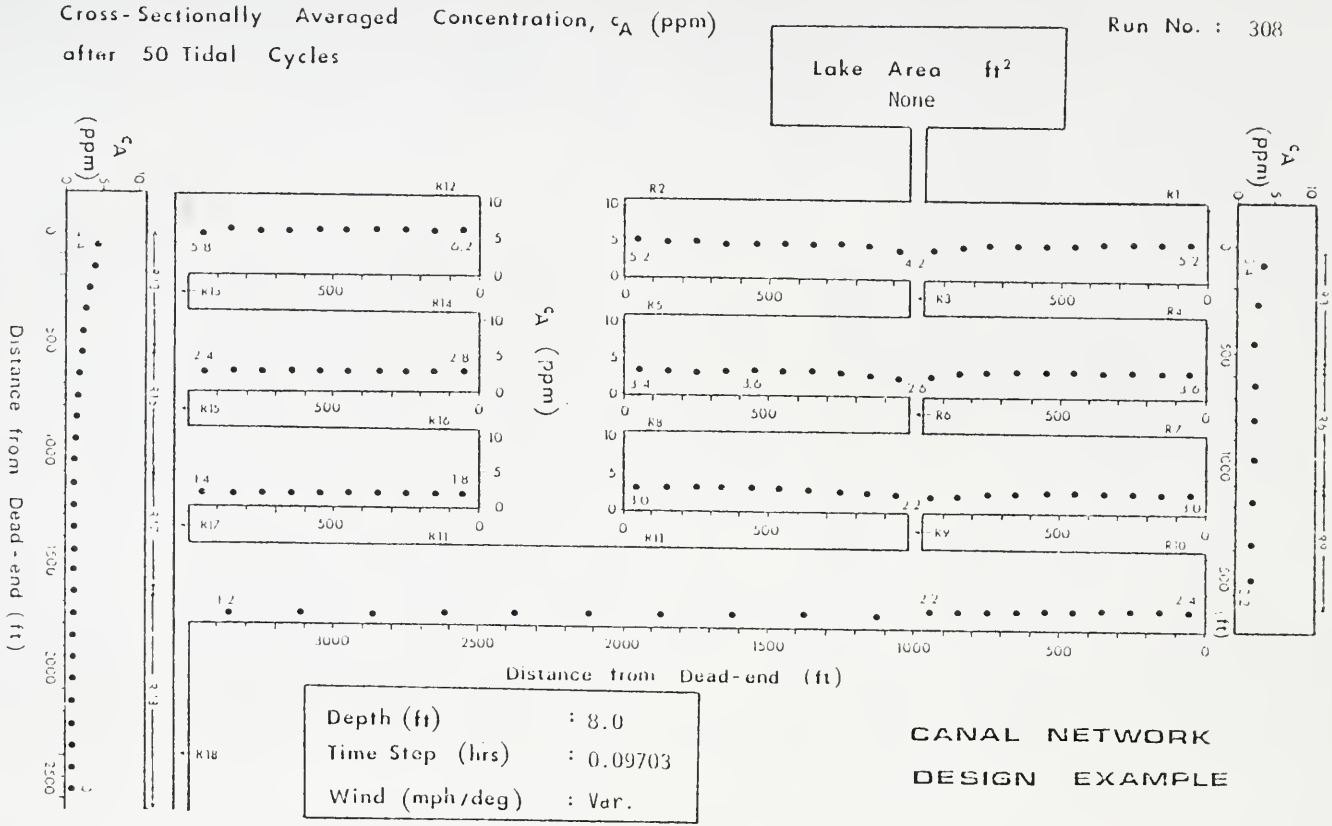


Figure 9.19 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With a Variable Wind Speed and Direction and No Lake (Results of Run No. 308).

CHAPTER 10

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

10.1 Summary

Residential canal systems are not inherently bad. If properly designed they can be aesthetically pleasing, environmentally compatible, biologically productive, and self-maintaining. The problem with many residential canal systems in the southeastern part of the United States is that they have been forced into the coastal landscape, rather than designed and constructed to function harmoniously with nature. Too often the desire or need for a fast economic turnover of investment, a lack of adequate capital, or a lack of regional planning and governmental control have resulted in the construction of canal systems which have destroyed large areas of wetlands and other coastal features of high natural value. Also, many of these canal networks have been inadequate for handling the increased loading of wastes which accompany a development. The construction of a hydrosystem which does not fit in with natural conditions at the site invariably produces a maintenance burden for the residents, which cannot be overcome until such canals are reconfigured by man or by the slow but inexorable processes of nature.

There are many different possible symptoms of trouble in canal systems, the causes of which will depend upon the natural characteristics of the site and the canal network itself. Water pollution is a commonly

cited symptom often associated with fish kills, odors, unattractive color and hazards to human health. Sedimentation and the need for frequent dredging are a common complaint. Saltwater intrusion into the aquifer has become more and more a public issue, particularly in heavily populated areas of Florida. The loss of highly productive intertidal marshes and large portions of estuaries has been documented as one of the most serious effects of improper canal development. There are also effects on land: interruption of surface drainage patterns, alterations to or losses of basic components in natural ecosystems and the increased burden of human society on areas which do not have the carrying capacity for such development.

An understanding of the basic forces and mechanisms which affect and control tidal canal systems is the starting point for rational design. Hydrodynamics is the science which provides the basic theory of water movement and its effect in spreading suspended and dissolved substances in the water. Engineering provides the means for measuring conditions at a given site, analyzing the significance and interaction of these conditions and applying physical theory to obtaining realistic designs. The biological sciences provide the important understanding of the interrelationship and tolerances of the living parts of the canal ecosystem with the physical parts. Geology helps to explain the nature of some of the materials which form the boundaries of the canals and the basic structure of the land on which they are constructed. An appreciation of the contributions that are available from all of these disciplines, and a means whereby these descriptions can be integrated together to ensure that a canal system is environmentally compatible and self-maintaining, is an important aspect of canal design.

The canal design project has concentrated on the development of the fundamental structure of a canal design capability. This basic structure consists of the measurement techniques for assessing the characteristics of proposed sites and existing canal networks, a hydrodynamic model for simulating the mass-transfer in a canal network, a structured approach to the development of canal design criteria and constraints, and techniques for the analysis of proposed canal networks.

The importance of effective field surveys in connection with the development and application of predictive models has been demonstrated so often by other researchers that it should need no particular emphasis here. Good field measurements are vital to the success of any numerical modeling study, not only for describing the simultaneous variations of key hydrodynamic and mass-transfer indicators, but also for calibrating and verifying the model on a particular site. The consistency of the results of two out of the three comprehensive field surveys, conducted in June, 1977, at the Loxahatchee River canal and in July, 1977, at the 57 Acres site, and the general agreement of the results of the third study, in October, 1977, at the 57 Acres site, is evident. In all of these cases one value for the vertical momentum transfer coefficient, N_z, the vertical dispersion coefficient, K_z, and the background dispersion coefficient, E_o, provided the best fit of simulated concentration profiles. The June, 1977, field survey at the Loxahatchee River canal provided data which could be used for fitting the saline wedge model to salinity measurements and velocity profiles, using the dispersion parameters obtained in the 57 Acres verification tests.

The development of the hydrodynamic model began with a review of the applicable equations for tidal flow and dispersion of conservative

substances. Early in this development it became apparent that in Floridian canals with small tidal ranges and very low tidal velocities, the discharge through a channel could be described analytically by assuming that the water surface will be horizontal throughout the network at all times. That this is a reasonable assumption was shown by comparisons using a model which simulated the small surface slopes associated with tidal flows in canals. The resulting mathematical expression for the mean tidal velocity at any cross-section in the network was found to be a function of the surface area upstream of that section, which is dependent on the location of that section in the network and time.

The development of a suitable numerical model for canal design and analysis began with a simple one-dimensional finite-difference digital model. It did not take long to realize that such a scheme has serious limitations when applied to the simulation of dispersion of a substance in fields of very small velocities. Various other classical approaches, such as the method of characteristics, sophisticated approaches, such as the method of second upwind differencing with flux-corrected transport, and the limited but powerful hybrid computer approach, were compared and analyzed. These models suffered from numerical dispersion to various degrees, which can sometimes be of the same order of magnitude as the natural dispersion that is being modeled. However one technique, the method of second moments, together with a flexible cell structure, resulted in a scheme with very small numerical dispersion. This method, in addition, is quite economical from the viewpoint of computer time, even in three dimensions.

In the method of second moments five different coefficients are used to specify rates of diffusion, dispersion, and momentum. These

coefficients can be related to known and observed physical processes in the canals, and used to calibrate the model for a given location. The numerical scheme was incorporated into a procedure for modeling networks of branching channels. The channels, or reaches, are joined by junctions which serve to link reaches of the same or different geometry. Large waterbodies, called "lakes," can be simulated with large junctions, and two tidal entrances may be included in a model under certain limiting conditions. The effect of wind on convection and dispersion is included by superimposing a depth-dependent, wind-induced velocity profile, determined analytically. Saltwater intrusion into a canal, and its effect on a substance in the water, can be included if the saltwater intrusion is in the form of a wedge.

The stability criteria for the model have been established. It has been calibrated, as mentioned above, with data from three comprehensive field surveys, and verified for results from a field study by EPA on Big Pine Key. The variablity of the model has been tested on straight, prismatic channels and compared with results obtained from earlier one-dimensional models.

The design of a new canal system usually begins with a selected site, or a search for a suitable site. Before the specific design work can be started, however, there are many planning considerations and decisions to be made. Some of the development and legislative considerations are summarized, and types of overall design objectives that can be developed to assure compatibility with both the developer's economic limitations and the legislative constraints that apply in the site's locality are illustrated. A method is shown by which these overall design objectives can be refined into specific design guidelines,

criteria, and constraints, with examples from publications by federal and state government, local planning agencies, a builder's organization, conservation interests, and a canal design engineer.

The predesign process also must include a preliminary site investigation. This consists of one or more visits to the site by the canal designer, the owner, and experts in various applicable natural sciences, to assess the characteristics of the site. Preliminary measurements of specific site characteristics are also required for the planning process. The types of information needed for planning and additional sources for relevant historical information and published data are outlined herein.

If an existing canal system is to be improved, several field surveys will also be required to obtain specific measurements of the hydrodynamic and water quality characteristics at, and in the vicinity of, the site. The requirements of such surveys are described in terms of data, instrumentation and support equipment, as is the reduction and analysis of such data.

A canal network can by synthesized from discrete design elements, consisting of sets of finger canals, as well as from individual branches and junctions. It is, therefore, advantageous to the designer to have an understanding of the operation of various configurations of networks which can then be linked together to form a larger network. One useful design element is the "comb-structured" group, consisting of two or more parallel, dead-end finger canals connected at one end by a main channel running at ninety degrees to the fingers. The flushing of this design element for no wind and for upstream and downstream winds was compared for a comb-structure without a lake, a comb-structure with a lake at the

end of the farthest reach from the tidal entrance, a comb with bending, instead of straight, finger canals, and a system with two tidal entrances and a null point synthesized from two basic comb networks.

Finally, a simple canal network consisting of a single set of comb-structures and a double set of comb-structures, with one tidal entrance, was used to show how the depth and size of the lake, if any, can be varied to optimize network flushing under various fixed and variable wind conditions. Relative flushing times, at the junction and the dead-end which had the highest concentration after fifty tidal cycles, were plotted and then extrapolated assuming first-order decay, where appropriate. Flushing of a point source under variable wind conditions was demonstrated.

10.2 Conclusions

Along the southeastern and Gulf Coastline of the United States there are areas which should not be disturbed in any way, and there are areas which are ideally suitable for properly designed residential canals. Those areas which are not tolerant of development are generally the low areas, the wetlands, which are most useful to man in their natural state. An exception may be a wetland which has already been altered from its highly productive, natural state, or one that has been created by dredge spoil and is only marginally useful. The areas which normally are suitable, provided they do not naturally perform some other more important function, such as providing aquifer recharge, irreplacable wildlife habitat, or storm protection, are usually uplands. An area may be intuitively judged for its unsuitability for development by the amount of modification (removal of vegetation, earth moving, filling for foundations) which would be required to complete the work.

All aspects of a residential canal development should be planned ahead by considering the environmental, legislative, and engineering constraints on the project as early and as completely as possible. Suggested design guidelines from a number of sources are included, most of which express some specific aspect of the general guideline that the design should fit into the natural characteristics of the site and function harmoniously with natural processes. A procedure for developing general objectives into specific design criteria and constraints is presented to ensure that in the process of moving from the general planning process to specific design, no important steps will be overlooked.

It has been recognized that field work comprises two important and somewhat different tasks. The first is the preliminary site investigation, in which the site characteristics are assessed and preliminary knowledge for planning subsequent field surveys is gained. The field survey work itself should be planned considering, first, the type of information desired from the analysis; second, what parameters are to be measured, and when; and third, how the variables are to be measured or sampled. The types of instruments and support equipment which have been found successful for collecting the necessary data have been described.

The results of field surveys, in particular the three comprehensive field surveys conducted during the last year of the project, are summarized and discussed in detail. It is concluded that field work of this nature is vital to a canal design and must be planned and executed efficiently and with great care. The following specific conclusions can be drawn with respect to the instrumentation and procedures required for canal design work:

1. Measurements of vertical velocity profiles are necessary for determining the structure of the flow in a canal if it is stratified, and for measuring bed shear stress. However, if the flow is tide- and wind-induced only, and a reasonable estimate of equivalent sand roughness can be obtained by other methods, velocity measurements may not be required for the design.
2. Background dye concentration should be measured in the canal network and in the receiving waterbody at least once, and preferably more than once, to determine its variability, before the tracer is released.
3. The mixing of standard tracer solutions and the calibration of the fluorometer should be thoroughly rehearsed before the field trip, and a record of the actual mixing procedure should be prepared for the data file.
4. Samples of dye at the approximate peak value which is expected to be prevalent in the canal during the experiment should be prepared and suspended in pyrex bottles at the sampling depths. The concentration in these bottles should be measured daily.
5. The method of continuous dye concentration recording along the centerline of the canal should be changed to a sampling process at suitable cross-sections rather than a continuous measurement. At least one sample should be outlined in each of the three

layers at each cross-section, and one to each side at the quarter-width points would be desirable if enough boats, equipment, and personnel are available.

The numerical model has been shown to be a unique design tool which enables the canal designer to perform variability tests economically and with a variety of input conditions. It has been developed with a minimum number of calibration coefficients, and the geometry of a test canal network can be changed easily.

The layout of a trial canal network on a topographic map of a site is a highly subjective process. The initial decisions will involve location of the tidal entrance(s) and alignment of the major channels in the system so that the available site area is used efficiently as well as aesthetically and in a way that will result in desirable hydrodynamic characteristics. Consideration of wind characteristics at the site is a *principal* concern at this stage of design. The following design principles relating to the wind should be followed:

1. To ensure adequate flushing of the entire network, the *finger* canals should be aligned in the direction of the prevailing wind during the season with the mildest wind condition.
2. If all finger canals are not to be aligned in the same direction, those farthest from the tidal entrance are most difficult to flush and therefore should be aligned as in number 1.) above.
3. Space should be made available, if possible, for a basin or "lake" near the farthest side of the canal

network from the tidal entrance. If the additional tidal prism that would result from the connection of a lake to the network is found to be necessary to ensure adequate flushing in the absence of winds, the surface area of the basin will usually need not be greater than about 50 percent of the surface area of the network.

As a result of a series of tests on the response of a comb-connected network design element to different wind conditions, the following conclusions can be drawn:

1. Tidal flushing alone in Floridian canals is not effective in large networks due to the small amplitude and limited excursion distance of the tides. In particular, the velocity near dead-ends is negligible and spreading of substances in this area can only occur by diffusion and wind-effects.
2. The effects of very small wind speeds, on the order of 2 mph or less, on canal networks is negligible.
3. The effect of a higher steady wind speed, on the order of 5 mph, is to completely change the convective patterns and the flushing characteristics in any portion of a canal network which is aligned with a wind component of 5 mph or more. In this case, reaches with downstream winds flush the fastest, and reaches with upstream winds still flush more rapidly than with tidal action alone, due to the establishment of vertical circulation by the wind.

4. The addition of a small basin or "lake" at a remote location in the network from the tidal entrance provides effective flushing throughout the connecting channels, and to some extent up into finger canals. However, the increase in velocities in these channels may cause the model to exceed its stability criteria, bed and bank as well as numerical, in which case the time increment may have to be reduced, or the spatial increment increased.
5. Reaches with bends are known to provide better vertical mixing than straight reaches. Since a network with bends must be modeled in three dimensions, and since the overall improvement in flushing which is contributed by bends is relatively slight, it is recommended that all network design be performed with two-dimensional (horizontally-averaged) simulations.
6. Symmetric networks with two tidal entrances can be analyzed provided that a null point can be located in the system. No significant improvement in flushing was noted in comparison with the single entrance comb-connected system, even for a 5 mph wind. A phase lag in the tides at the two entrances, which can be accomodated to some extent in the model, will be more effective in inducing tidal flushing then the case in which the tides are in phase at the entrances.

A hypothetical "existing" small network was taken as an example and a variety of simulations were run to observe the effects of changes in mean depth, wind, and tidal prism. It was found that the rate of flushing at dead-ends of finger canals is determined to a great extent by the wind, and in the absence of wind the traditional comb-structured canal will not flush effectively with a typical Floridian tidal range (2 ft) and depth (8 ft). It was shown that a decrease in depth improves tidal flushing to some degree, but there was no change in the flushing rate beyond the tidal excursion distance for tidal circulation alone.

Tests with steady winds blowing over the hypothetical canal network showed that a small wind velocity (2 mph) had little effect on flushing, while a somewhat higher wind (5 mph) had a much more significant effect. Comparing the effect of changes in depth with the 5 mph wind superimposed showed that a simple relationship does not hold over the range of 4 to 12 ft mean depth. It was concluded that for the limited range of variability in these simulations, an 8 ft mean depth provided the best canal flushing characteristics.

The effect of adding a "lake" to a remote junction in the network was shown to provide effective flushing in the main channels between the trial entrance and the lake, and a decreasing effect with distance into the finger canals.

10.3 Recommendations for Future Research

Additional development in a number of areas encompassed by the canal design research project would be possible. Ways in which the three-dimensional numerical model could be improved and expanded include a more general method for handling loops, particularly the case in which flows diverge and then recombine in an upstream reach, and the ability

to model more than one substance at a time, together with their interactions.

Several projects related to field work should be undertaken. One should be a study of the variability of Nikuradse's equivalent sand roughness in Floridian canals and the effects of different channel configurations (bends, various shapes and sizes of roughness elements, and junctions with branch canals, for example) on the magnitude of this variable. Another would be the development of better sampling devices, and possibly sampling arrays, for efficient measurement of dye concentration at many points in a canal. The development of a less expensive means for measuring dye concentration, one that would permit multiple continuous sampling stations to be established along a reach and avoid the need for sampling from a boat, would be extremely useful. Also, the photochemical and temperature-induced decay of Rhodamine WT should be evaluated to relate the rate of decay to a small number of measurable variables that could either be measured on site or predicted from published climatological data.

It will be necessary to define the range of applicability of the model and to quantify the relative error to be expected in its results over that range. For example, it has been found that the horizontal water surface assumption results in predictions of depth and velocity in a straight canal which are within 2 percent of the values predicted by a model which includes the momentum equation. These results were obtained for tidal amplitudes of 1 and 2 ft and lengths to 11,000 ft. It would be useful to extend this comparison, particularly with another two-dimensional model, to evaluate the error between the two sets of results as a function of the most important canal parameters.

The two- and three-dimensional modeling capability of CANNET3D now permits a realistic simulation of the flushing of tidal canal networks. It is apparent that dissolved oxygen modeling is still an elusive problem, because of the difficulty of defining sources, sinks, rates of decay, and the various coefficients (such as the reaeration coefficient) which have been traditionally used to attempt to predict this variable. It would probably be more realistic, at least until the DO/BOD balance in canals has been more thoroughly studied and realistically quantified, to use, instead, comparative predictions of the flushing rates of a conservative pollutant as the criteria for evaluating new and existing canal networks. A standardized approach to these simulations, therefore, needs to be defined and tested for a variety of cases.

Several different kinds of simulations have been illustrated in Chapters 8 and 9. These consisted of:

1. Constant pollutant inflow into the surface layer of the canal along all reaches, beginning at the concentration of the receiving waterbody.
2. No pollutant inflow, but all cells in the network initially set to an arbitrary value, and a comparison of the relative flushing rates at various locations in the network.
3. No pollutant inflow, and one high source of pollutant near a remote dead-end of the network.

Case number 1 is somewhat unrealistic, in that it is highly unlikely that a sustained, uniform, relatively high pollutant loading would occur over all of the banks of an entire canal network. New canal network designs now require an effective drainage plan with retention

or detention storage, so that at least pollutant inputs will be confined to a few point source locations. Case number 2 is an effective method for locating the slowest flushing points in the network, and could serve as the preliminary step in evaluating a network. However, it is not realistic in an absolute sense since a typical network would not be loaded uniformly, and flushing rates are very slow in this case because of the small concentration gradients in the network. Case number 3 is the most realistic of the three, but it will be necessary to consider the type, magnitude, and duration of the pollutant loading to be simulated in order to obtain realistic results.

Finally, it will be necessary to apply the model to many more actual cases in order to prove that it is representative of a wide variety of Floridian canal networks. Once more has been learned about the quantitative effects of changes in canal parameters on the flushing characteristics of these systems, and the information has been gereralized, cost functions could be developed for a design optimization procedure.

APPENDIX A

SALINITY, TEMPERATURE, VELOCITY

and

DYE CONCENTRATION MEASUREMENTS

LOXAHATCHEE NORTH CANAL

June 10-16, 1977

A.1

SALINITY, VELOCITY

and

DISPERSION STUDY

SALINITY, VELOCITY AND DISPERSION STUDY

Loxahatchee North Canal

June 10-16, 1977

Objective:

This study was conducted to obtain simultaneous data on water velocity, salinity and dye concentration which could be used to determine the movement of salinity currents and their effect, with wind and tide, on the dispersion of dye.

Tide Records:

LOW TIDE		HIGH TIDE	
Date	Time	Date	Time
770613	0230	770613	0730
	1430		2100
770614	0315	770614	0815
	1530		2130
770615	0400	770615	1530

Current Meters:

Electromagnetic current meters were installed at stations 1+50, 11+00 and 21+00.

Dye Injection: Time: 770613 1123

75 ml of Rhodamine WT 20% solution mixed with 3/4 gallon of canal water was released and mixed across the canal of the surface at station 2+00. Wind was measured at 3.9 mph from East.

Sampling depth: 3 ft

A.2

CROSS-SECTIONS AND PLAN VIEW

CLIMATOLOGICAL DATA, JUNE 1977

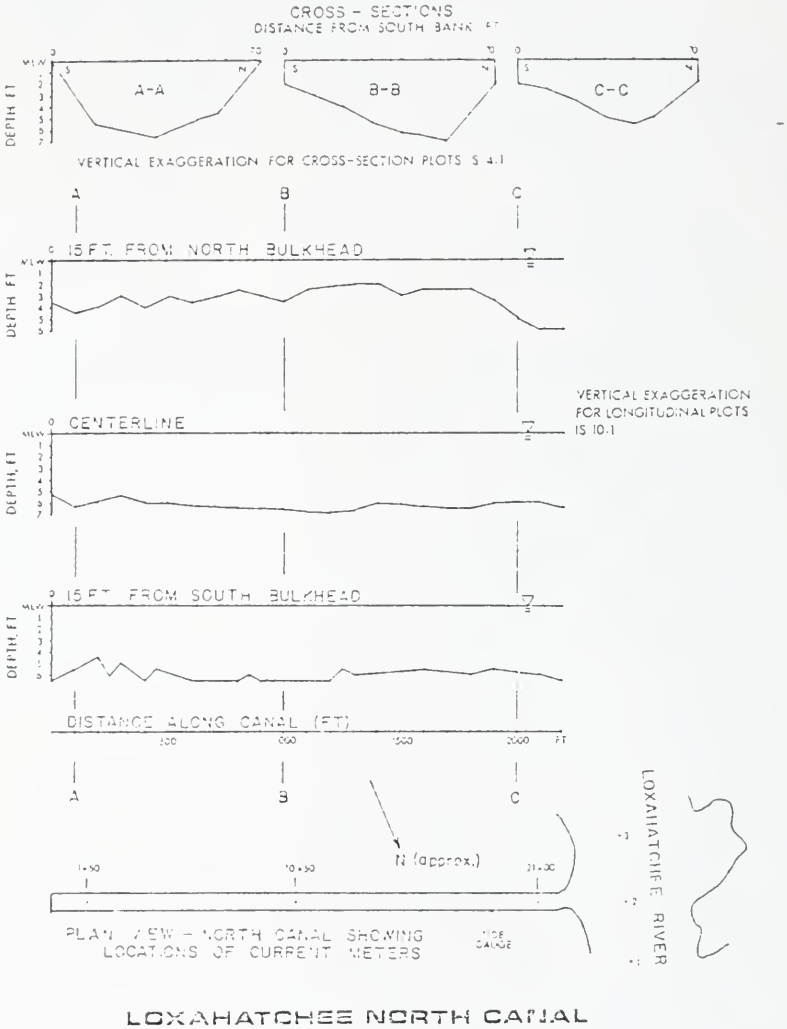


Figure A.1 - Cross-Sections and Plan View of Loxahatchee North Canal, June 1977.

OBSERVATIONS AT 3-HOUR INTERVALS

NOTES

CEILING

WEATHER

WIND

STATION

WEST PALM BEACH FLA

YEAR & MONTH

U.S. DEPARTMENT OF COMMERCE
NATIONAL CLIMATIC CENTER
FEDERAL BUILDING
ASHEVILLE, N.C. 28801

AN EQUAL OPPORTUNITY EMPLOYER

POSTAGE AND FEES PAID
U.S. DEPARTMENT OF COMMERCE

COM-210

Figure A.2 - Climatological Three-hourly Observations for June, 1977.

A.3

Vertical Profiles

of

Salinity, Longitudinal Velocity,

and

Dye Concentration

LOXAHATCHEE NORTH CANAL

June 13-15, 1977

This section contains plotted results as follows:

DATE	TIME SPAN	TIDE
770613	1249 - 1334	ebb
770613	1452 - 1750	flood
770614	1043 - 1259	ebb
770614	1424 - 1603	ebb
770614	1536 - 1758	flood
770614	1758 - 1858	flood
770615	1201 - 1447	ebb
770615	1550 - 1700	flood

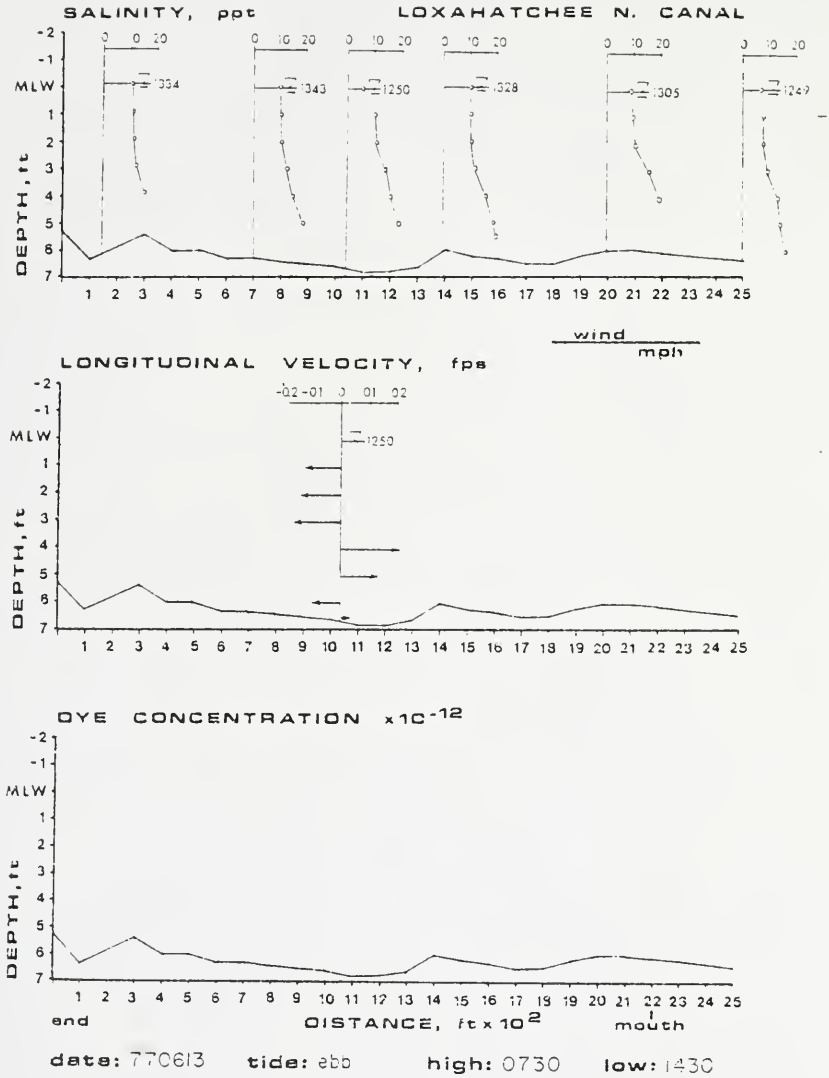


Figure A.3 - Vertical Salinity, Velocity, and Dye Concentration Profiles, Loxahatchee North Canal. (Date: 770613, Time Span: 1249 - 1334).

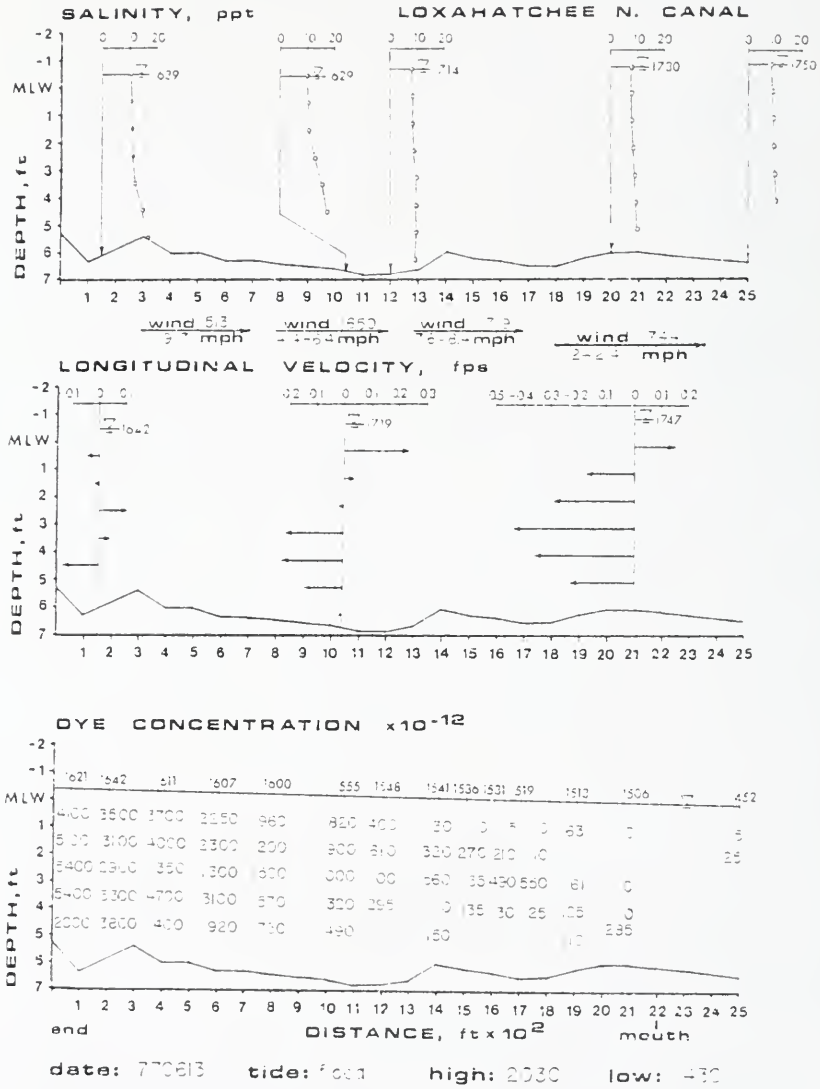


Figure A.4 - Vertical Salinity, Velocity, and Dye Concentration Profiles, Loxahatchee North Canal. (Date: 770613, Time Span: 1452 - 1750).

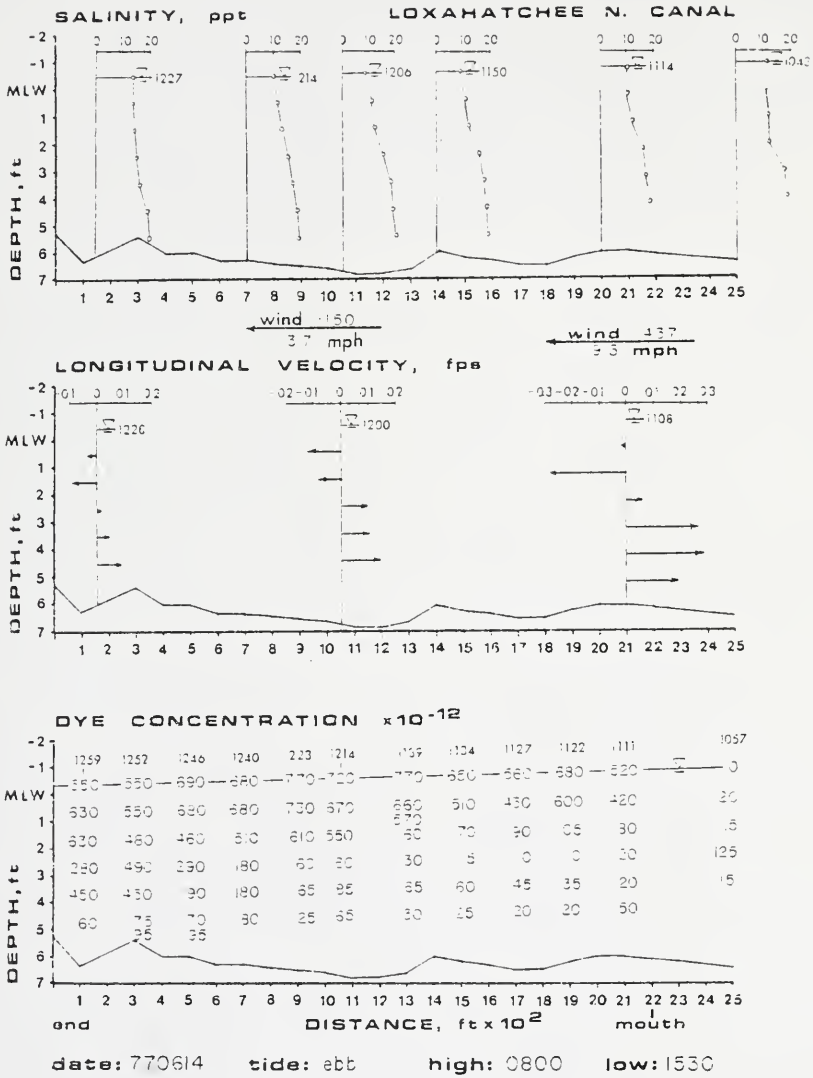


Figure A.5 - Vertical Salinity, Velocity, and Dye Concentration Profiles, Loxahatchee North Canal. (Date: 770614, Time Span: 1043 - 1259).

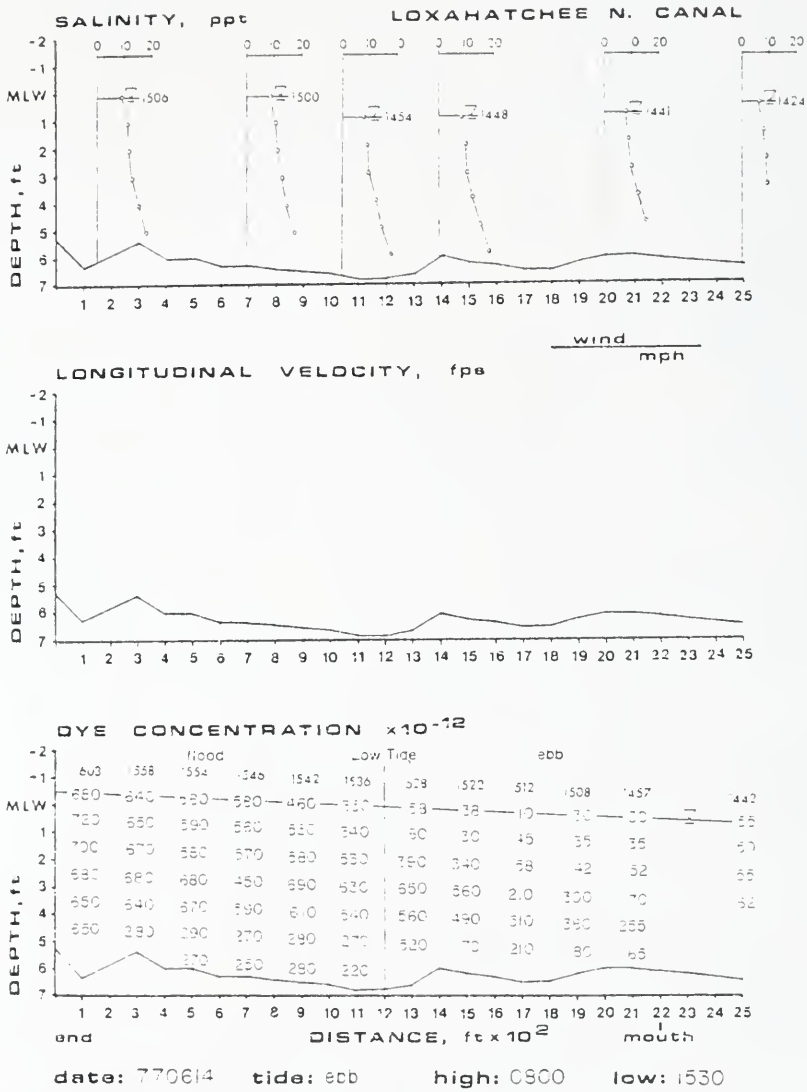


Figure A.6 - Vertical Salinity, Velocity, and Dye Concentration Profiles, Loxahatchee North Canal. (Date: 770614, Time Span: 1424 - 1603).

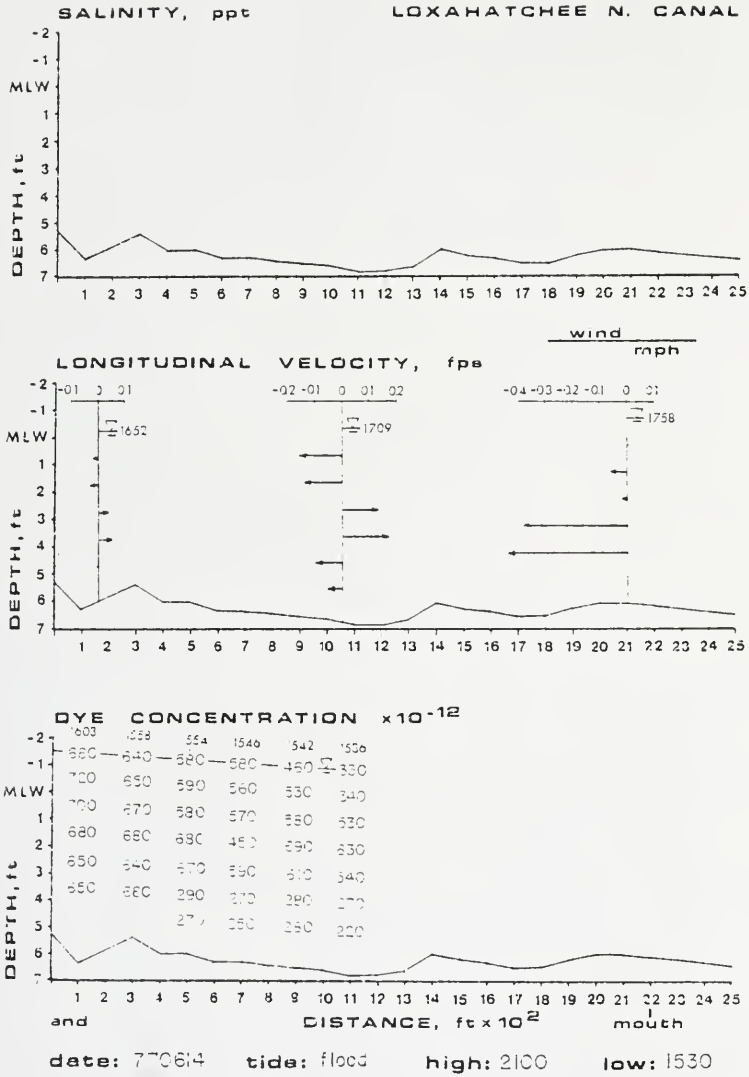


Figure A.7 - Vertical Salinity, Velocity, and Dye Concentration Profiles, Loxahatchee North Canal. (Date: 770614, Time Span: 1536 - 1758).

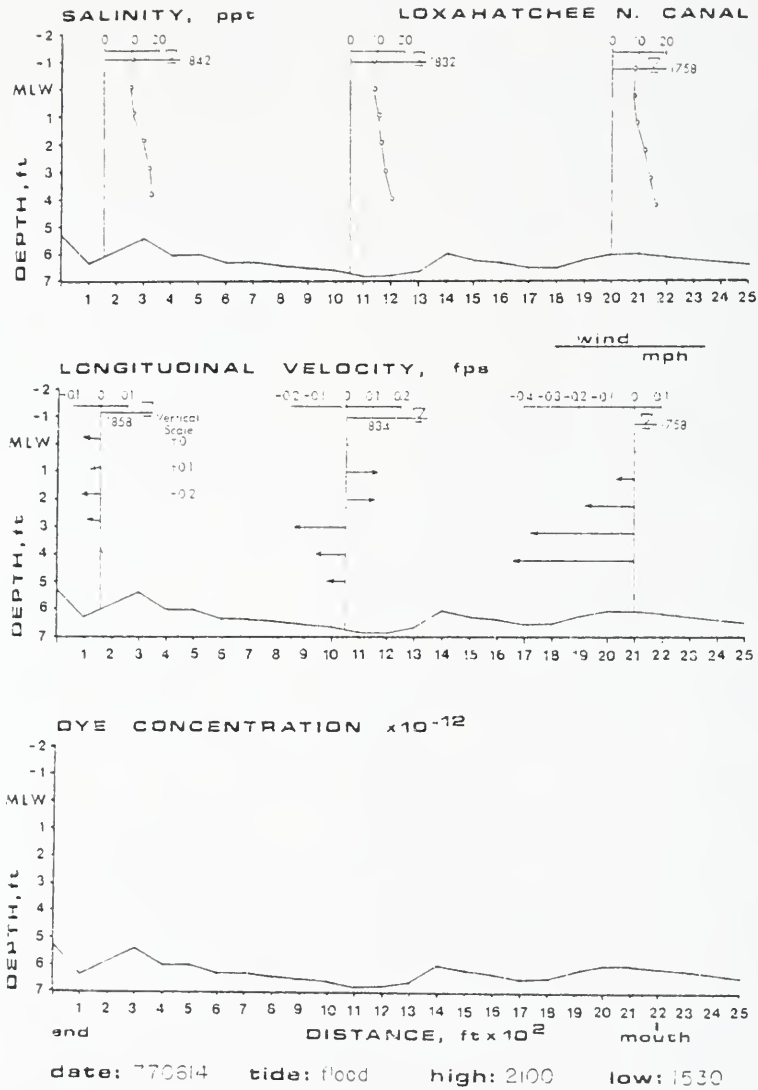


Figure A.8 - Vertical Salinity, Velocity, and Dye Concentration Profiles, Loxahatchee North Canal. (Date: 770614, Time Span: 1758 - 1858).

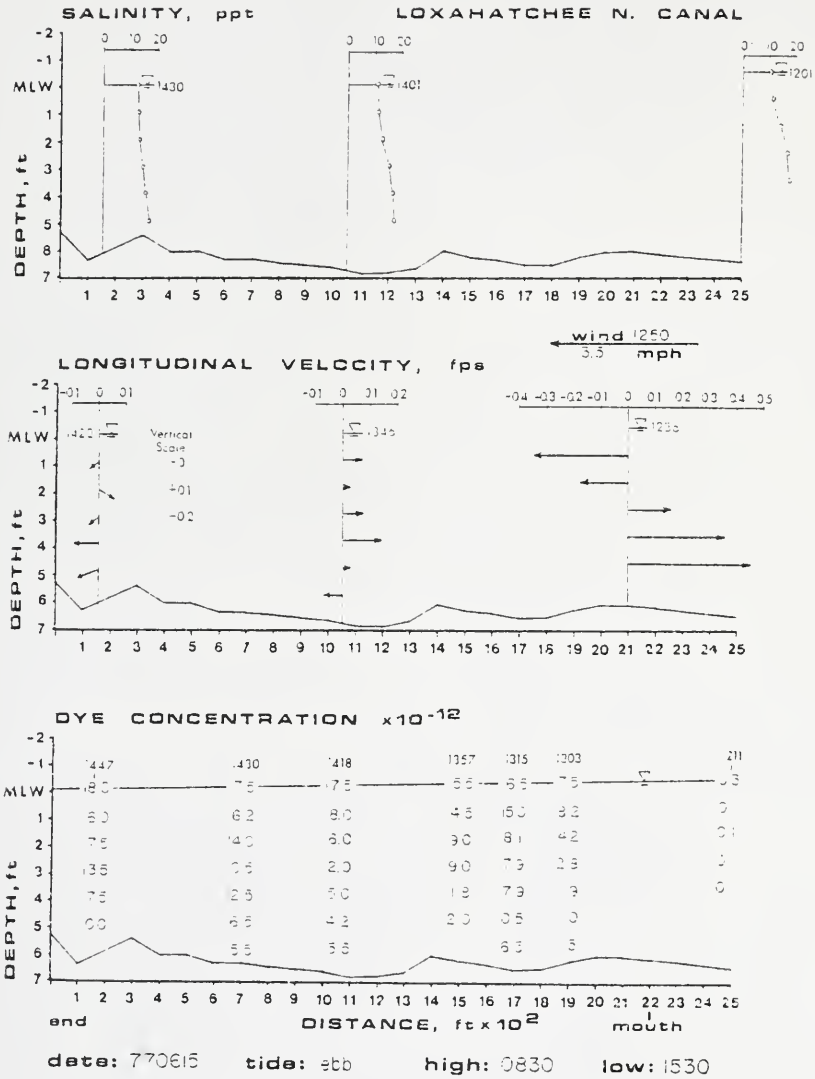


Figure A.9 - Vertical Salinity, Velocity, and Dye Concentration Profiles, Loxahatchee North Canal. (Date: 770615, Time Span: 1201 - 1447).

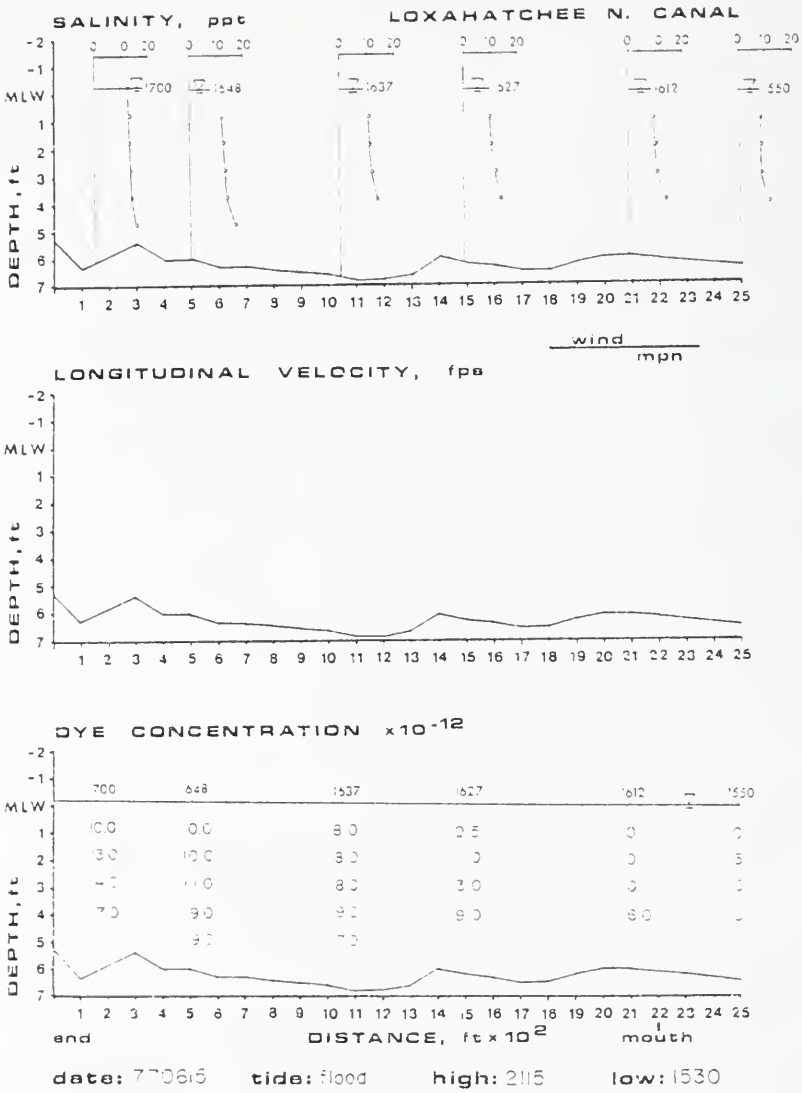


Figure A.10 - Vertical Salinity, Velocity, and Dye Concentration Profiles, Loxahatchee North Canal. (Date: 770615, Time Span: 1550 - 1700).

A.4

Vertically Averaged

Dye Concentration Measurements

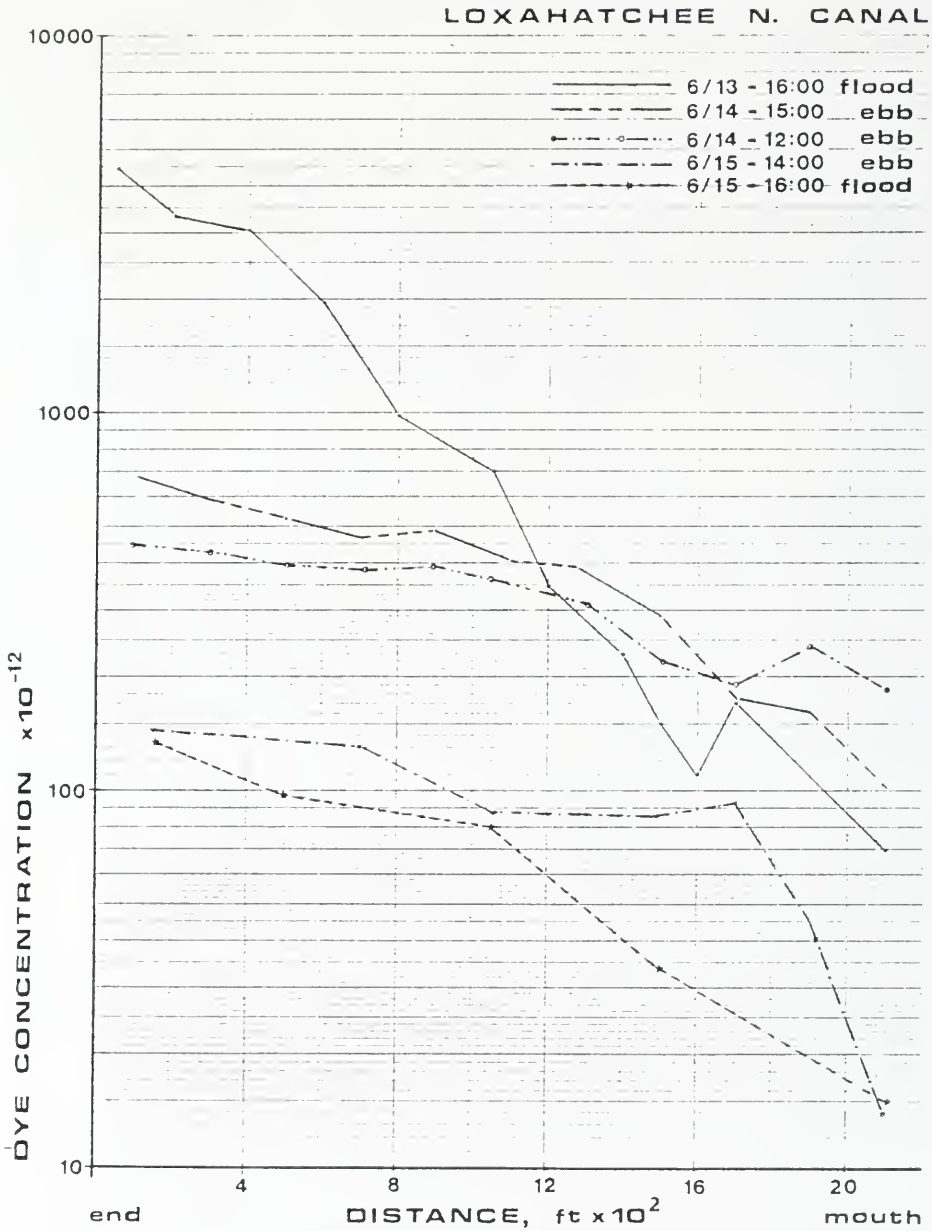


Figure A.11 - Vertically Averaged Longitudinal Dye Concentration Profiles.

APPENDIX B

LONGITUDINAL DISPERSION STUDIES

at

57 ACRES SITE

PALM BEACH COUNTY

July and October 1977

B.1

LOCATION MAP

CANAL NETWORK MAP

CANAL GEOMETRY

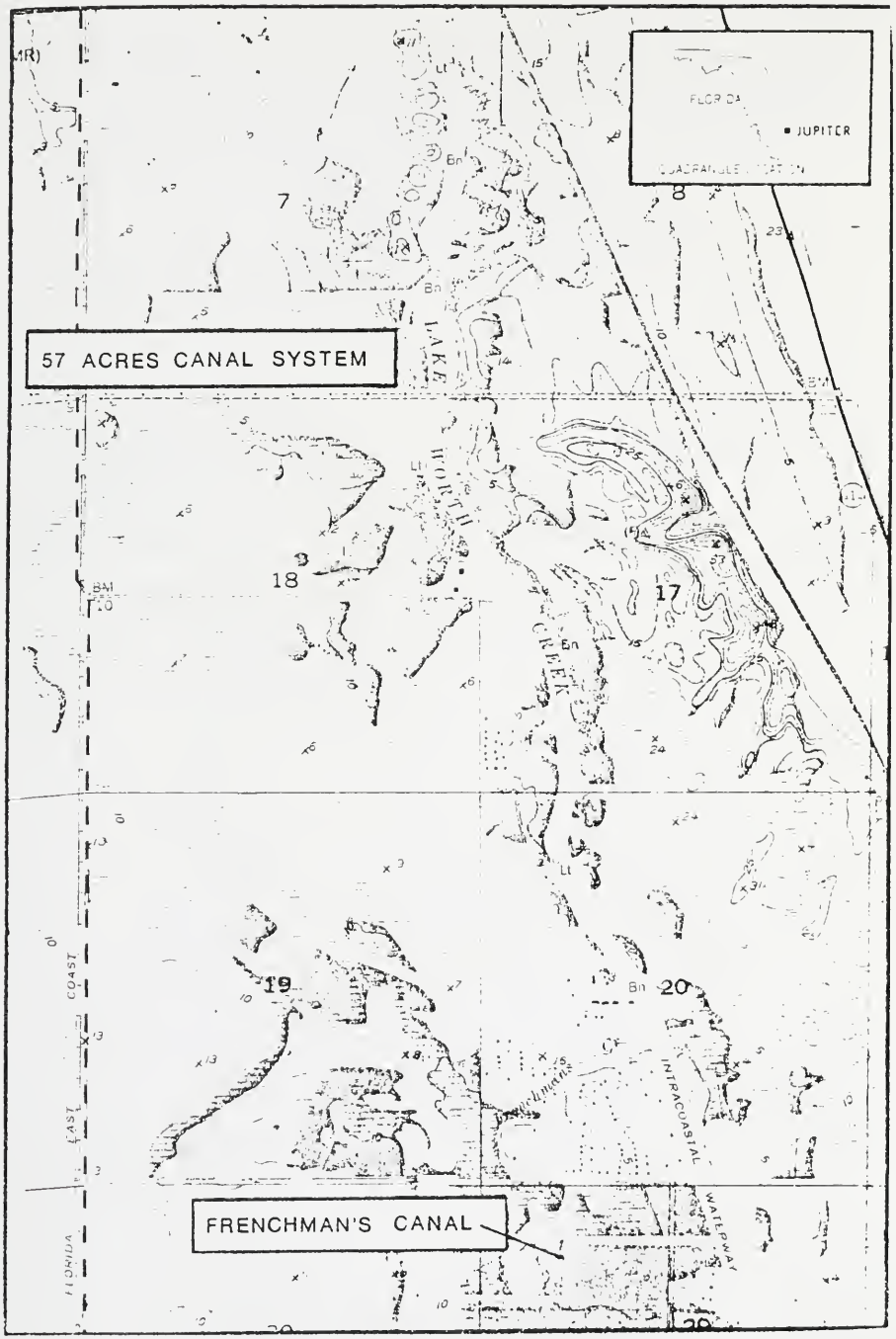


Figure B.1 - Location Map and Topography of 57 Acres Project Site.

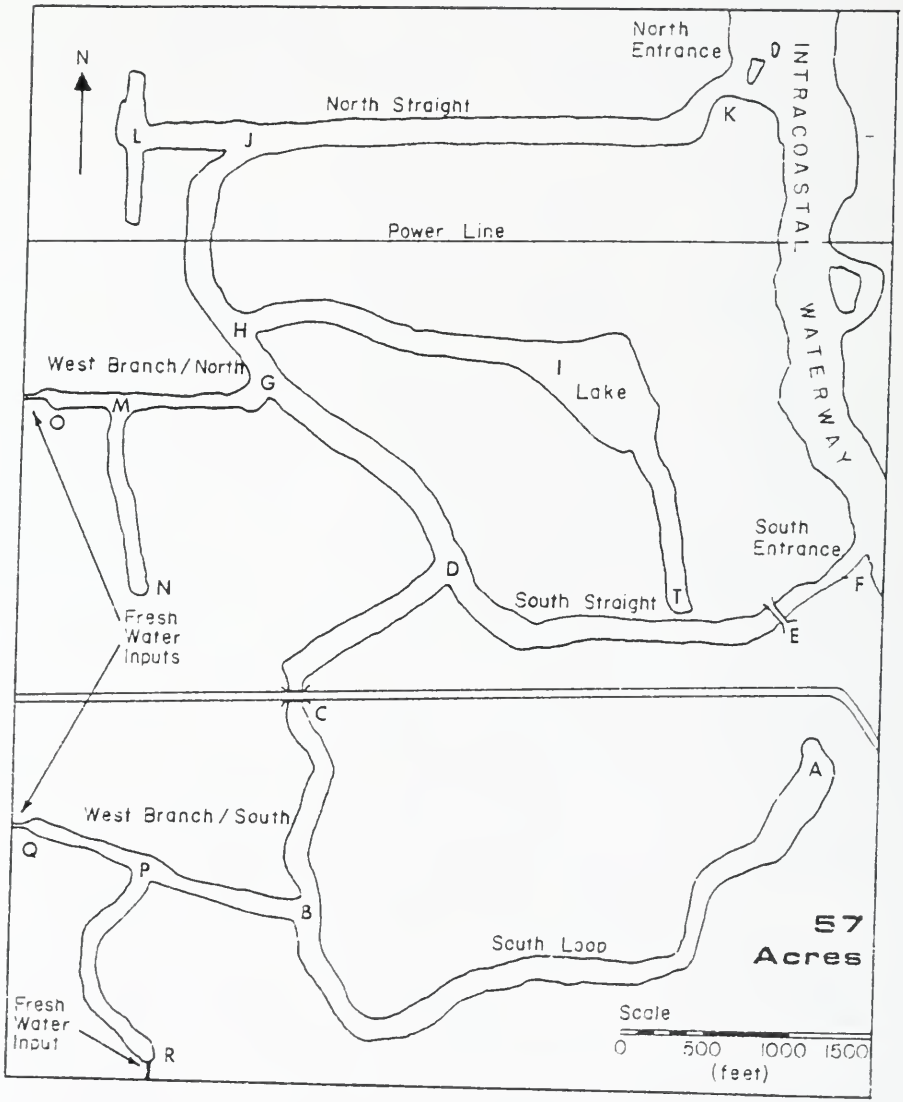


Figure B.2 - 57 Acres Site Plan Showing Bathymetry and Locations Used to Designate Reaches for Dye Dispersion Measurements.

Table B.1 - Lengths, Widths, and Mid-tide Depths of Canals in 57 Acres Network.

REACH	L	B	d_o
AZ	1000	180	11.6
ZB	3430	100	8.0
BP	870	120	8.0
PQ	730	100	6.5
PR	1370	90	6.2
BD	2625	110	8.4
DY	2040	120	9.0
YF	600	95	7.0
DG	1545	120	10.0
GM	830	95	9.0
MD	430	90	9.0
MN	800	70	8.5
GH	365	115	8.5
HI	1630	110	8.0
I	950	340	19.0
IT	970	100	9.6
HJ	1265	125	7.0
JL	670	125	8.0
JK	2760	125	8.0

B.2

First Dispersion Study

July 18-22, 1977

FIRST DISPERSION STUDY

57 Acres Project

July 18-22, 1977

Objective:

The first dispersion study was intended primarily as a test of the feasibility and utility of a centerline dye concentration measurement throughout the canal network over several tidal cycles.

Tide Records:

LOW TIDE		HIGH TIDE	
Date	Time	Date	Time
770719	1830	770720	0030
770720	0630		1230
	1935	770721	0105
770721	0715		1335
	1935	770722	0145
770722	0755		1405

Current Meters:

None

Dye Injection: Time: 770720 0040

1500 ml Rhodamine WT 20% solution released as point source about 400 ft from dead-end of south loop, station A. Total weight of dye was calculated to be 0.78 lb. Dye was released at surface from a bottle and mixed with paddle. There was no wind.

Dye sampling runs:

HIGH TIDE			LOW TIDE		
Start Time		Peak (ppb)	Start Time		Peak (ppb)
770720	1347	140	770720	1940	135
770721	0005	105	770721	0809	105
	1335	(Missed)		1933	42
770722	0222	52	770722	0807	50
	1402	45			

Sampling Depth:

3 ft

OBSERVATIONS AT 3-HOUR INTERVALS

NOTES

CEILING

WEATHER

WIND

STATION

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Figure B.3 - Climatological Three-hourly Observations for July, 1977.

B.2.1

FIRST DISPERSION STUDY

57 ACRES

July 19 through 23 1977

THREE-DIMENSIONAL MEASURED DYE CONCENTRATION PLOTS

This section contains plotted results as follows:

DATE	TIDE	TIME
770720	High	1230
770720	Low	1935
770721	High	0105
770721	Low	0715
770721	High	1325
770721	Low	1935
770722	High	0145
770722	Low	0755
770722	High	1405

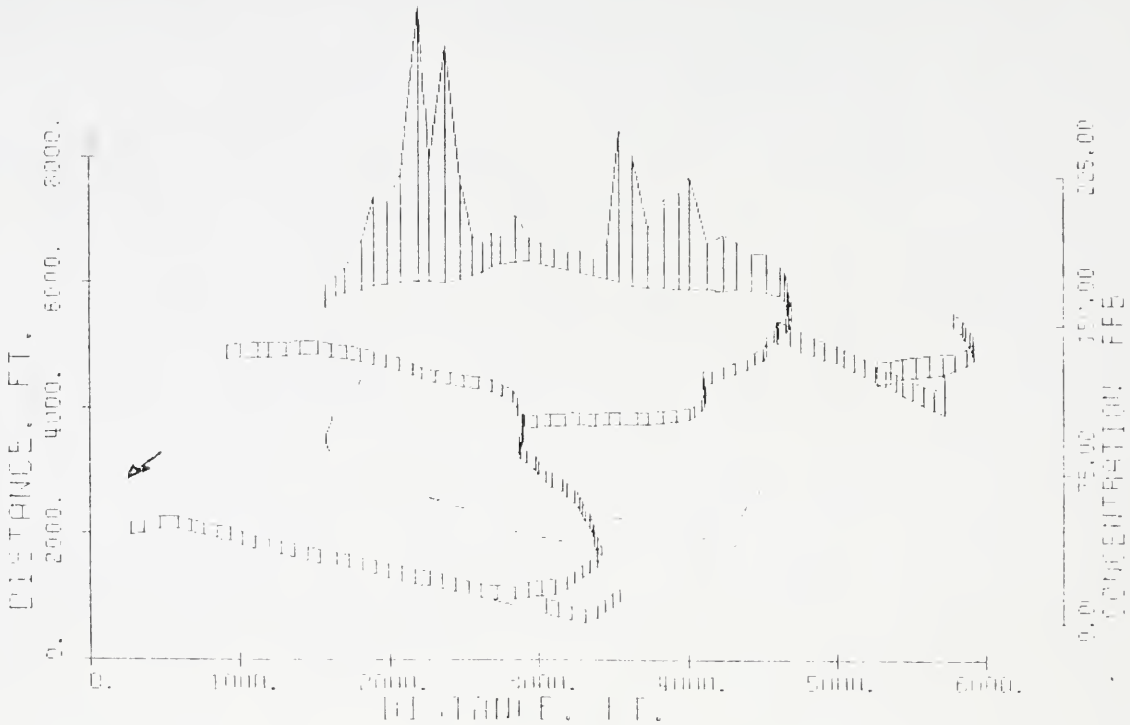

Figure B.4 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, Florida. Date: 770720. Tide: High. Time: 1230. Peak: 140 PPB.

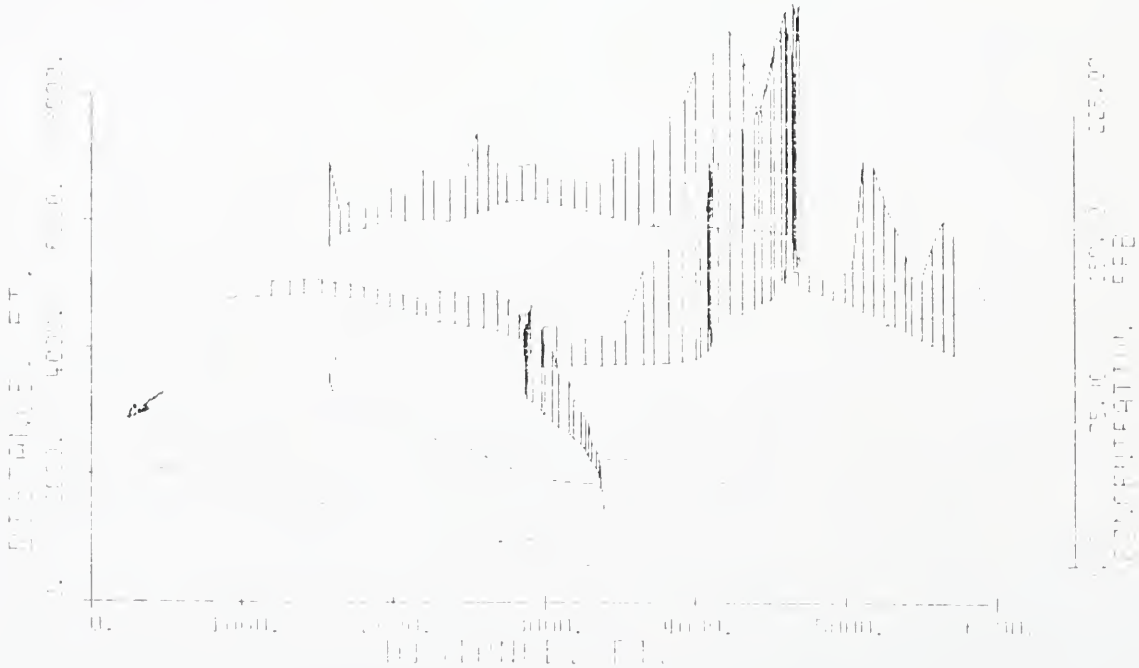

Figure B.5 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, Florida. Date: 770720. Tide: Low. Time: 1935. Peak: 135 PPB. Azimuth: 210 Degrees. Elevation: 25 Degrees.

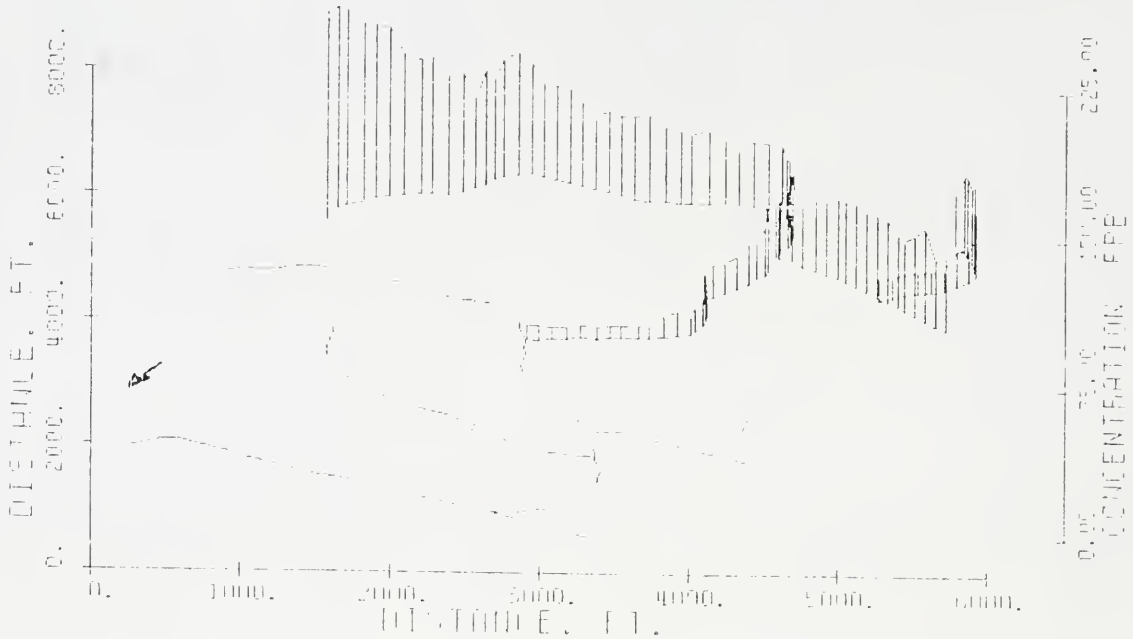

Figure B.6 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, Florida. Date: 770721. Tide: High. Time: 0105. Peak: 105 PPB. Azimuth: 210 Degrees. Elevation: 25 Degrees.

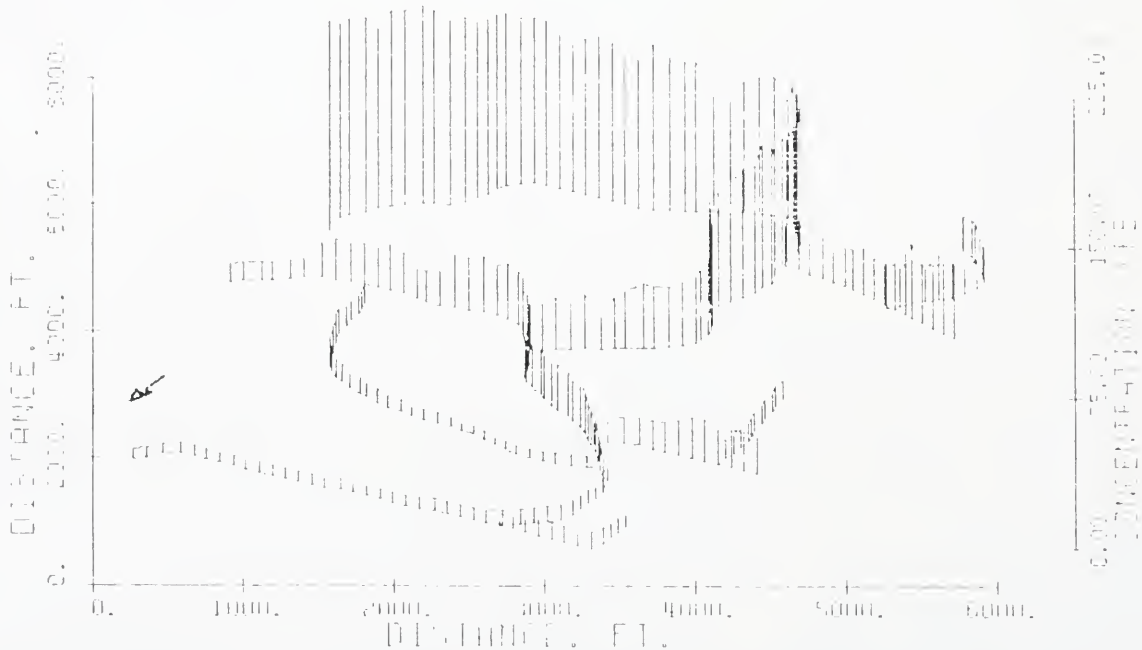

Figure B.7 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, Florida. Date: 770721. Tide: Low. Time: 0715. Peak: 105 PPB. Azimuth: 210 Degrees. Elevation: 25 Degrees.

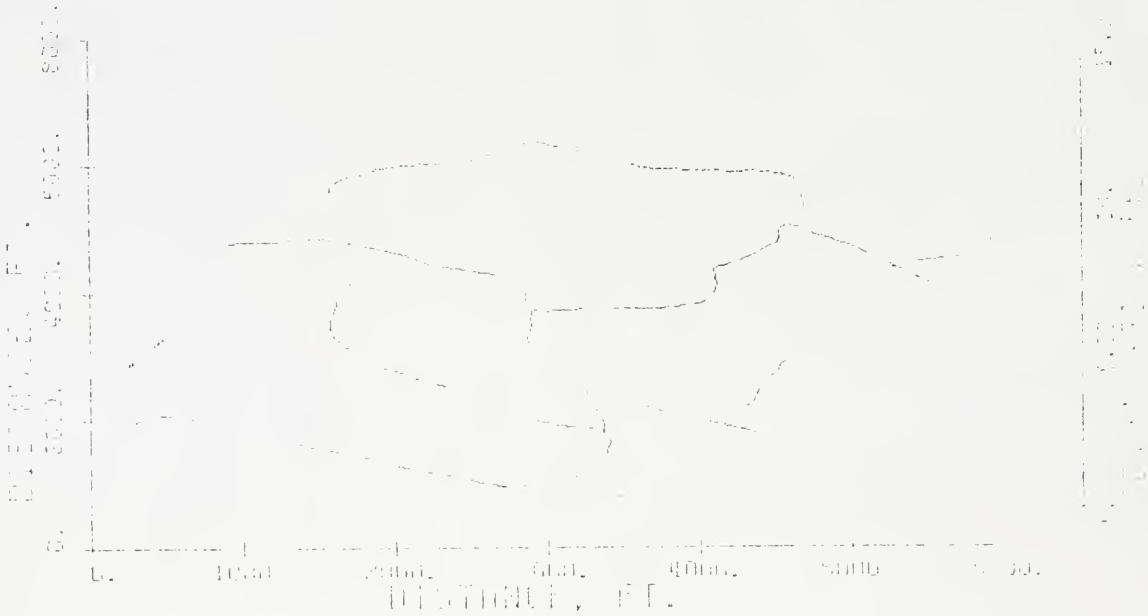

Figure B.8 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, Florida. Date: 770721. Tide: High. Time: 1325. Peak: No Data. Azimuth: 210 Degrees. Elevation: 25 Degrees.

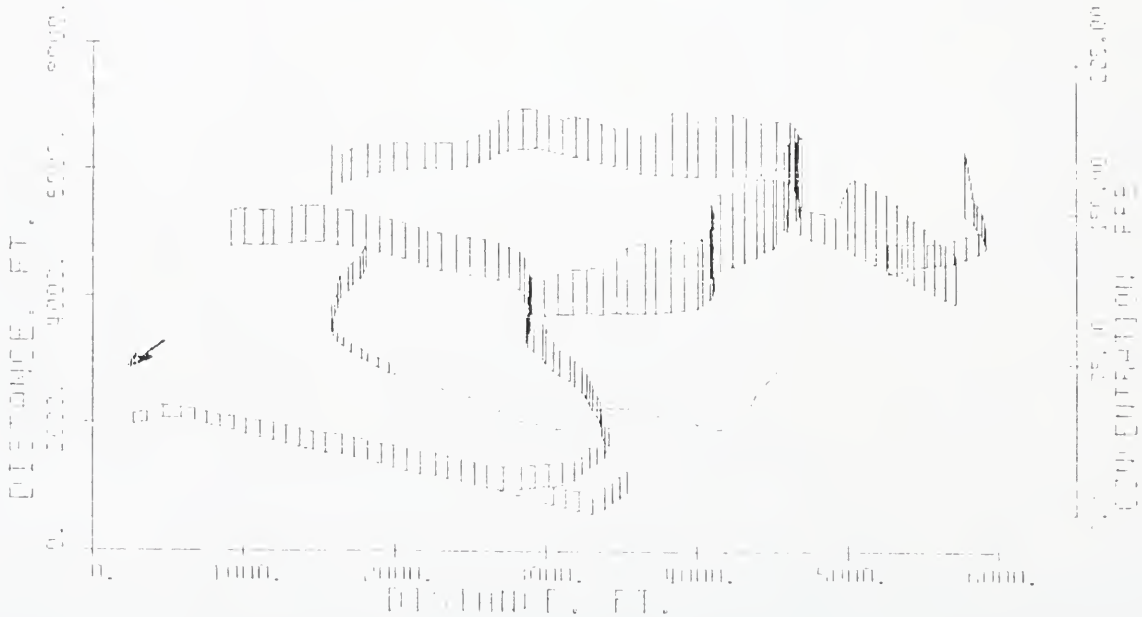

Figure B.9 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, Florida. Date: 770721. Tide: Low. Time: 1935. Peak: 42 PPB. Azimuth: 210 Degrees. Elevation: 25 Degrees.

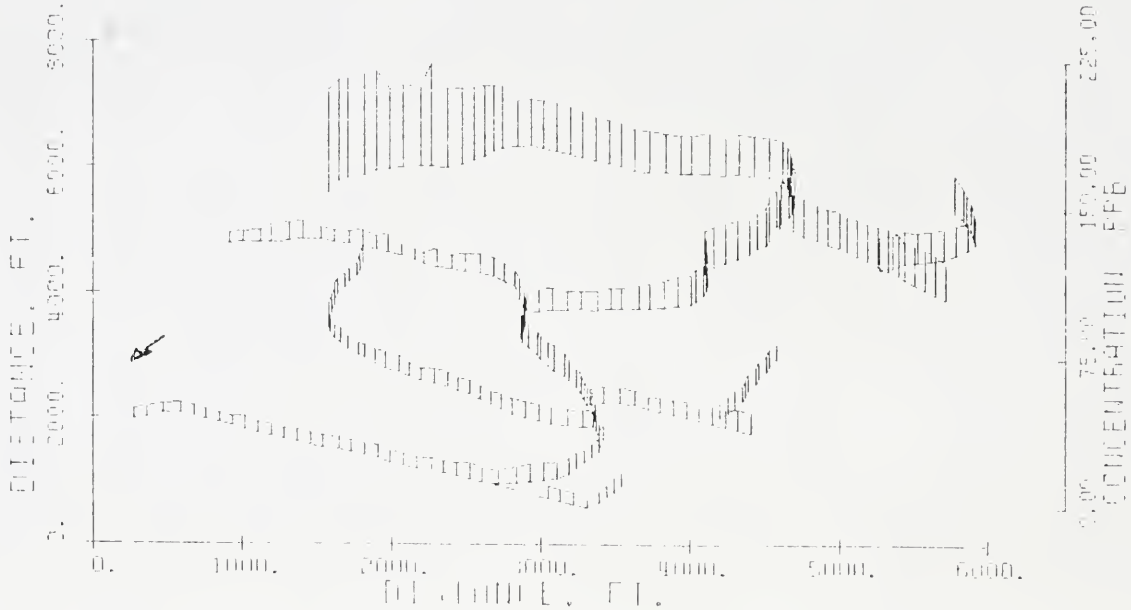

Figure B.10 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, Florida. Date: 770722. Tide: High. Time: 0145. Peak: 52 PPB. Azimuth: 210 Degrees. Elevation: 25 Degrees.

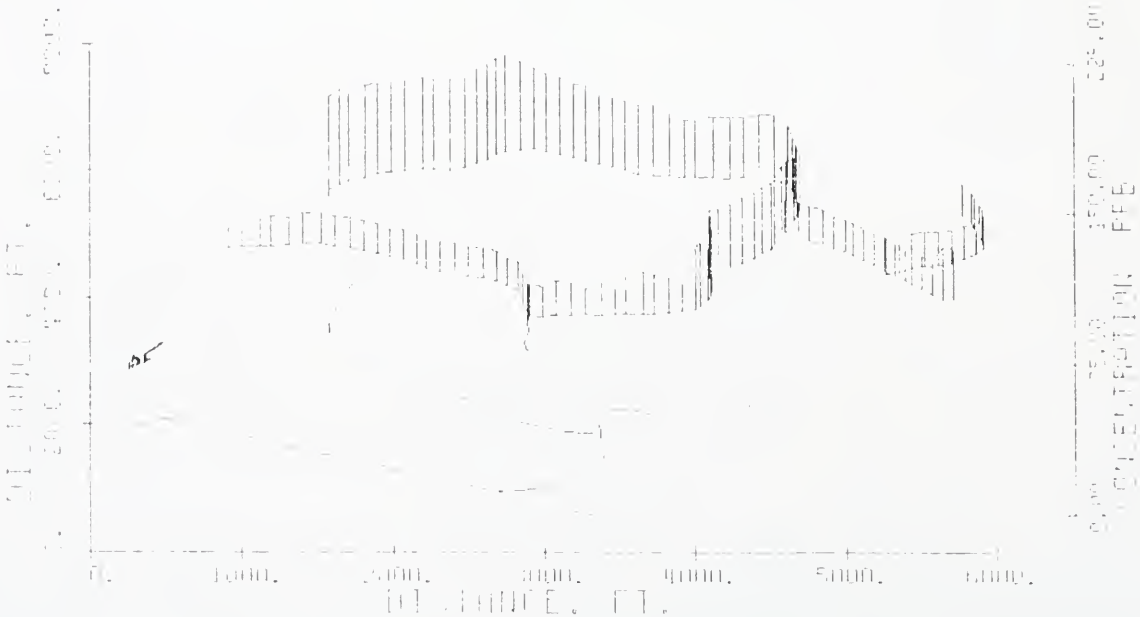

Figure B.11 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, Florida. Date: 770722. Tide: Low. Time; 0755. Peak: 50 PPB. Azimuth: 210 Degrees. Elevation: 25 Degrees.

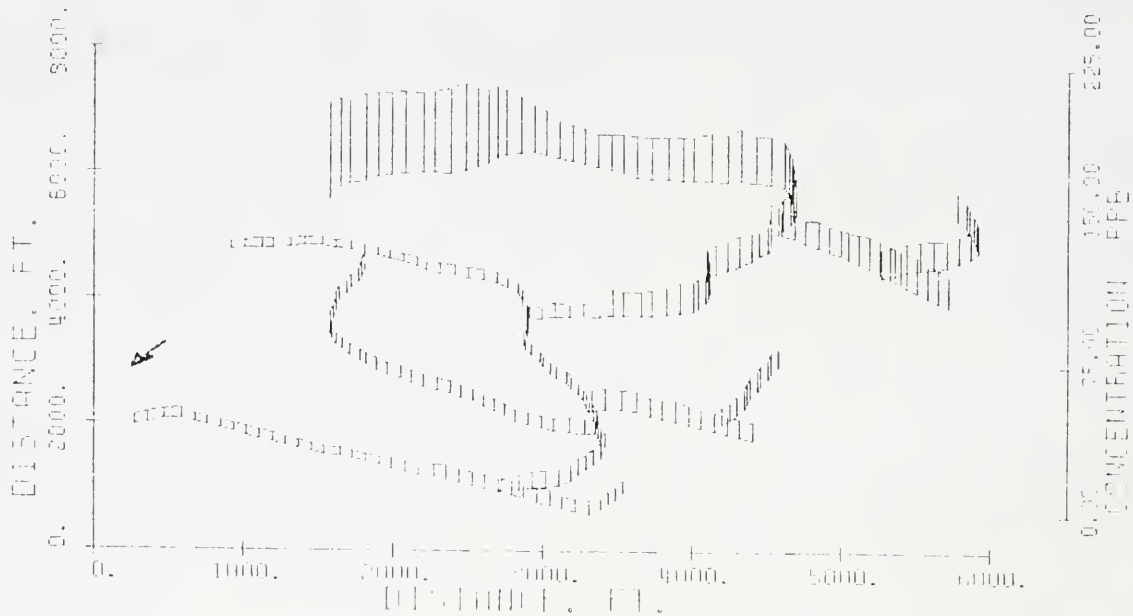

Figure B.12 - Measured Dye Concentration at 3 ft Depth Along Centerline. 57 Acres Canal System, Palm Beach County, Florida. Date: 770722. Tide: High. Time: 1405. Peak: 45 PPB. Azimuth: 210 Degrees. Elevation: 25 Degrees.

B.2.2

FIRST DISPERSION STUDY

TWO-DIMENSIONAL MEASURED DYE CONCENTRATION PLOTS

This section contains plotted results as follows:

REACH	TIDE
AD	High
AD	High

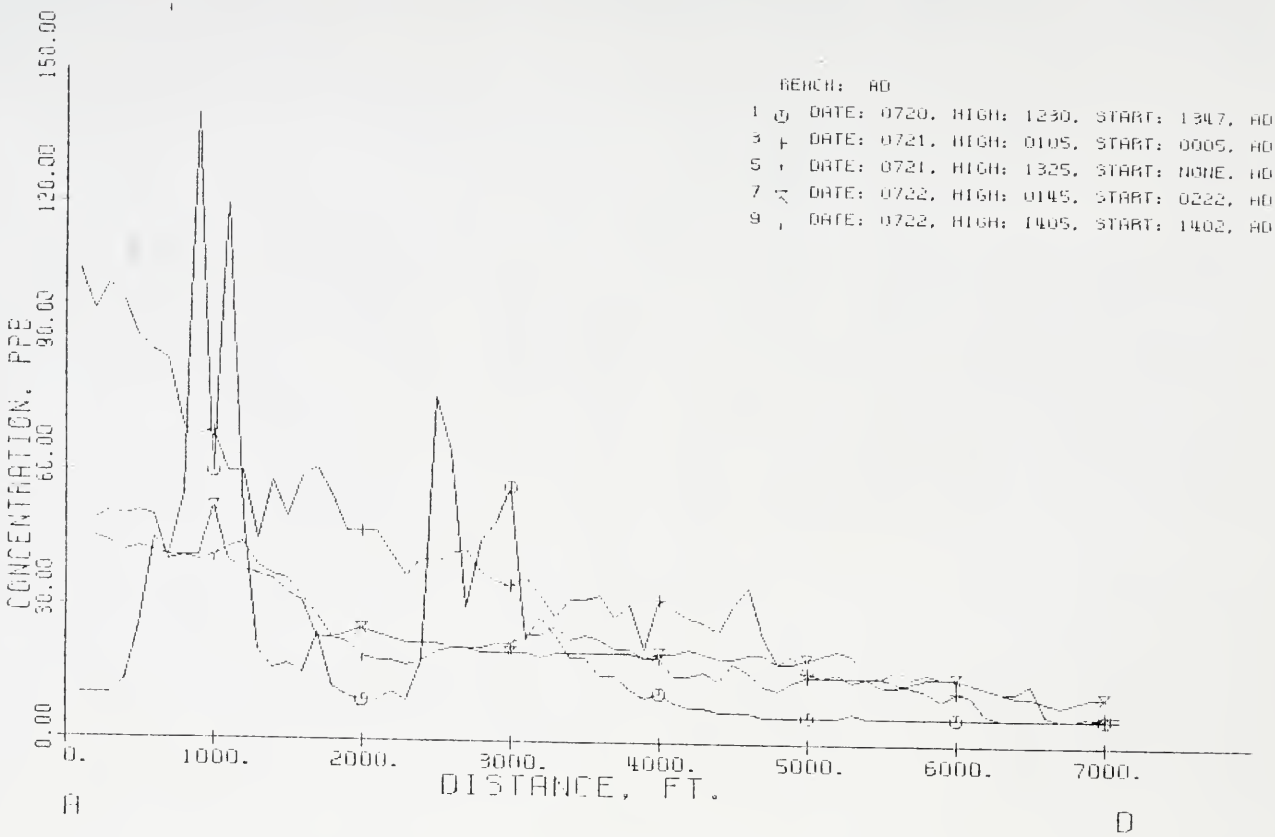

Figure B.13 - Measured Dye Concentration at 3 ft Depth along Centerline in 57 Acres Canal System, Palm Beach County, Florida, July 1977.

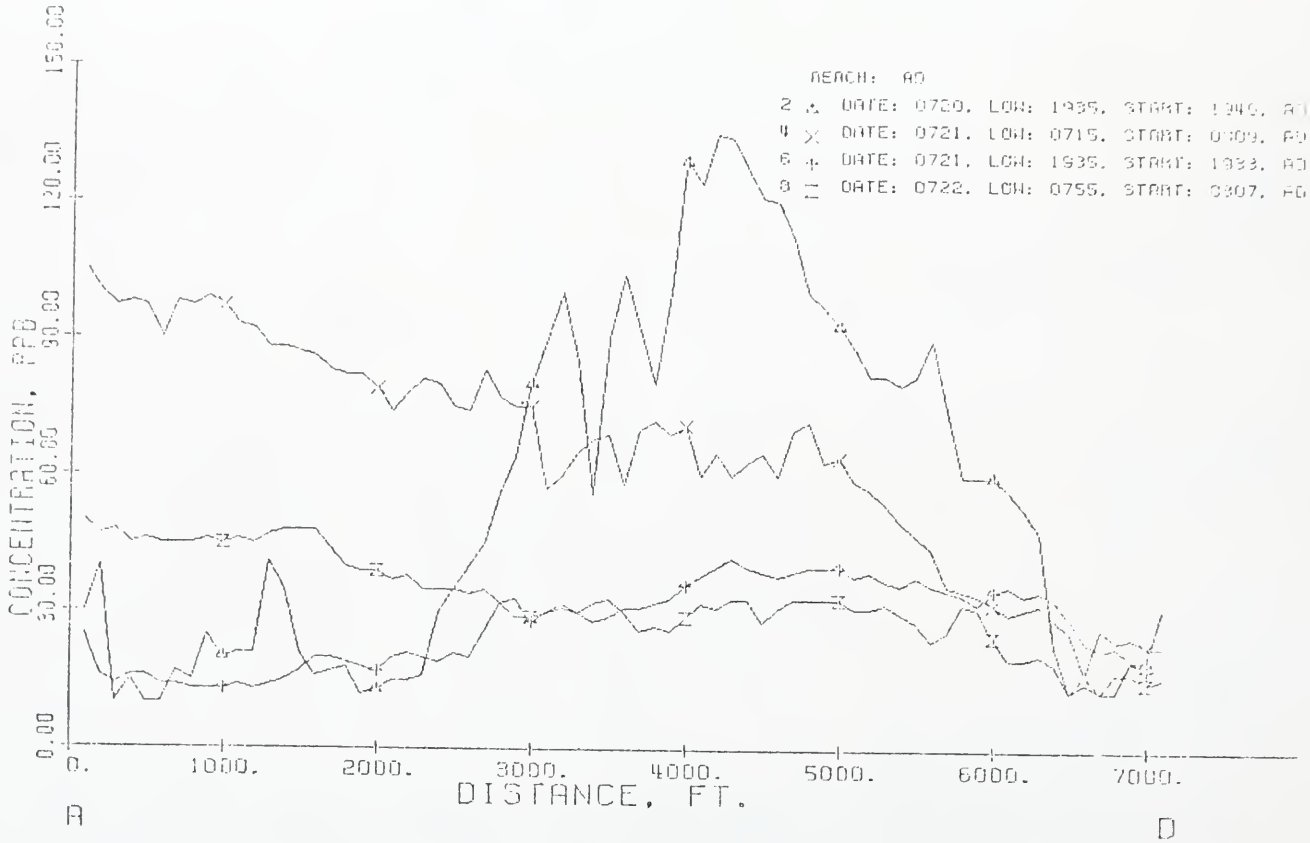

Figure B.14 - Measured Dye Concentration at 3 ft Depth Along Centerline of 57 Acres Canal System, Palm Beach County, Florida, July 1977.

B.3

Second Dispersion Study

October 16-22, 1977

SECOND DISPERSION STUDY

57 Acres Project

October 16-22, 1977

Objective:

The second dispersion study was intended primarily to obtain simultaneous measurements of tidal height, wind, water velocities, salinity, water temperature, and centerline dye concentration in order to relate dye dispersion to water circulation.

Tide Records:

HIGH TIDE		LOW TIDE	
Date	Time	Date	Time
771019	1500	771019	2140
771020	0330	771020	0950
	1610		2240
771021	0420	771021	1045

Current Meters:

Current meters were installed at stations 1, 2, and 3.

Dye Injection: Time: 771019 1500

1500 ml Rhodamine WT 20% solution released under pressure from a tank mixed with canal water at approximately 10-20 psi, through a 5/8 inch pvc pipe with three 1/4 inch holes spaced at 1 ft intervals from end and capped. Dye released in 3 ft surface layer on transect at tide gauge for approximately two minutes as boat was driven through dye cloud. Dye cloud was then mixed with compressed air and propeller for additional two minutes.

Dye sampling runs:

LOW TIDE

Start Time		Peak (ppb)
771010	2210	10.0
771020	1102	5.7
	2308	5.3
771021	1139	3.0

Start Time		Peak (ppb)
771020	0401	5.4
	1641	2.9
771021	0428	2.2

Sampling depth:

3 ft

B.3.1

TWO LOCATION MAPS SHOWING LOCATIONS OF CURRENT METERS

CLIMATOLOGICAL DATA, OCTOBER 1977

MEASURED WIND VELOCITY, OCTOBER 1977

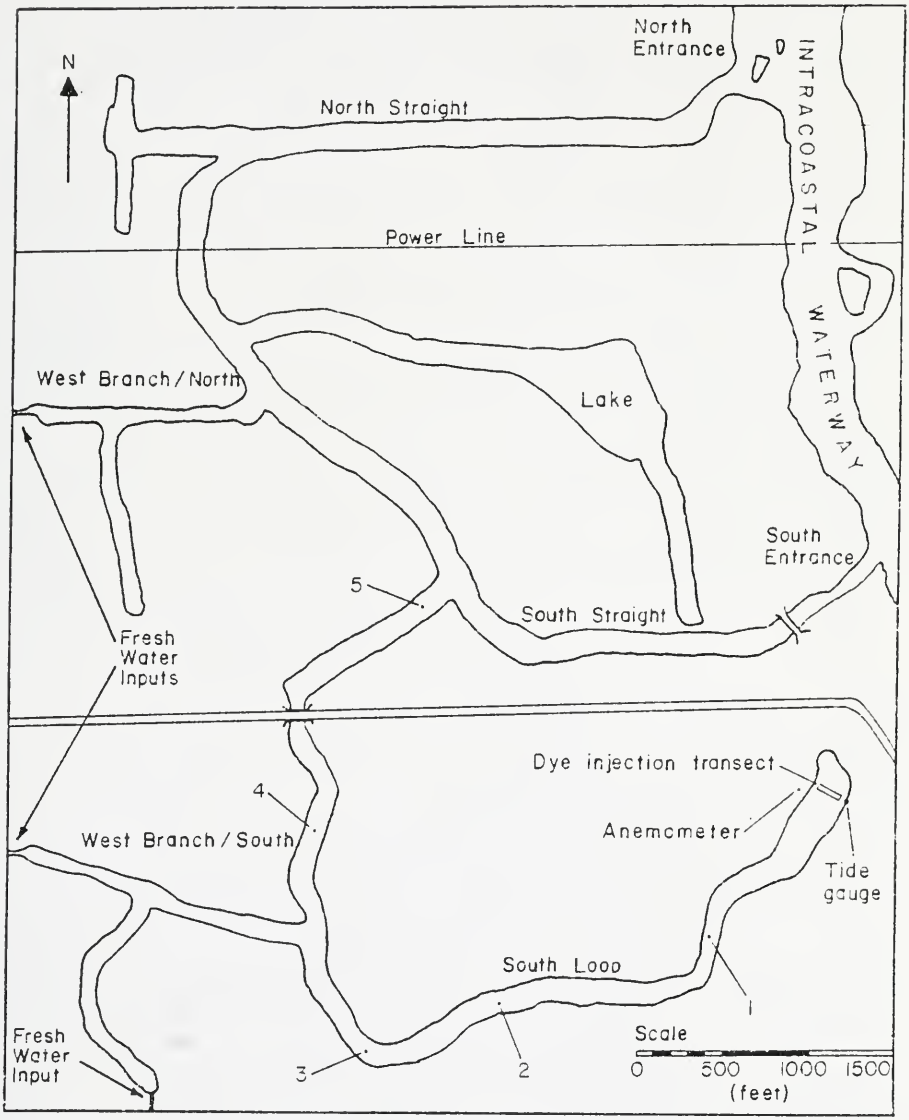

Figure B.15 - 57 Acres Site Plan Showing Location of Electromagnetic Current Meters for October 1977 Velocity, Salinity, and Water Temperature Measurements.

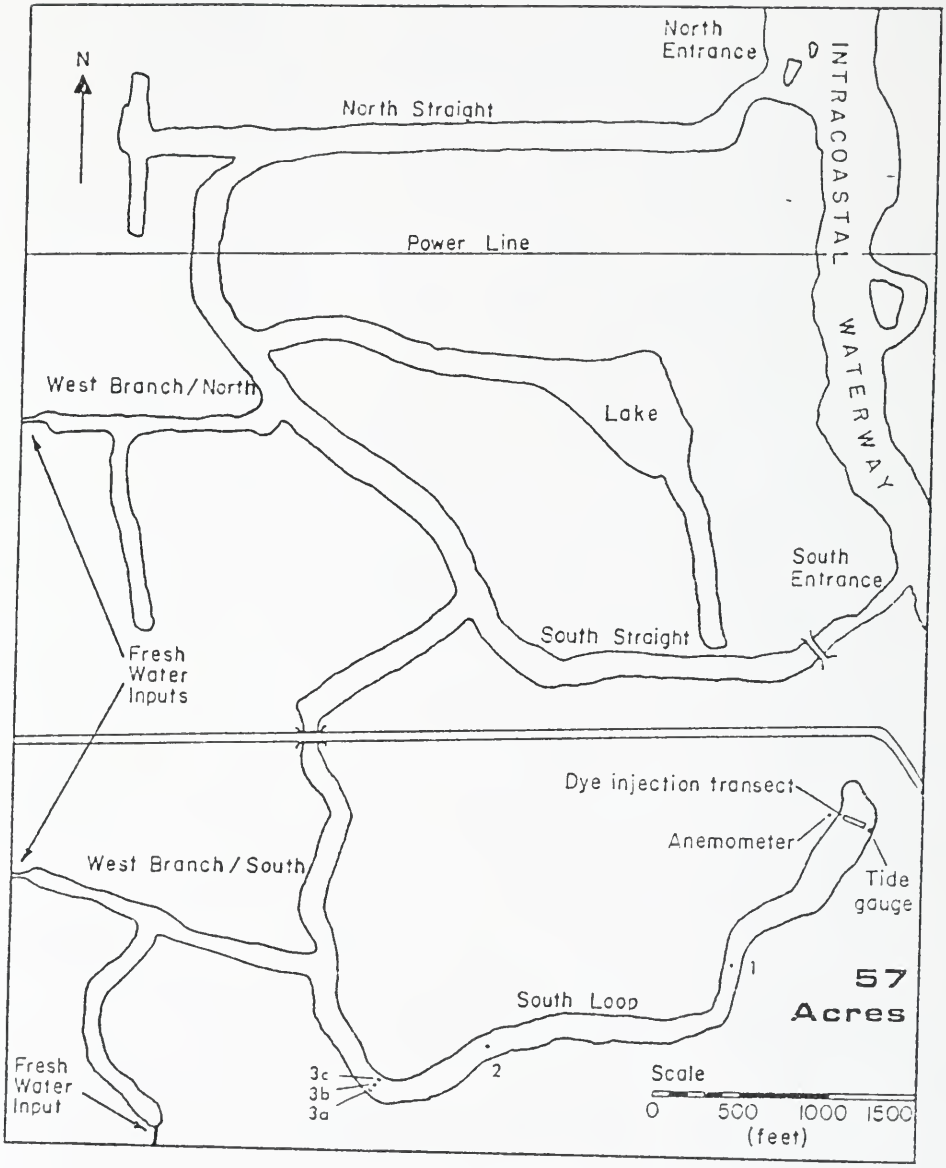

Figure B.16 - 57 Acres Site Plan Showing Location of Electromagnetic Current Meters for October 1977 Dye Dispersion Measurements.

OBSERVATIONS AT 3-HOUR INTERVALS

NOTES

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Figure B.17 - Climatological Three-Hourly Observations for October, 1977.

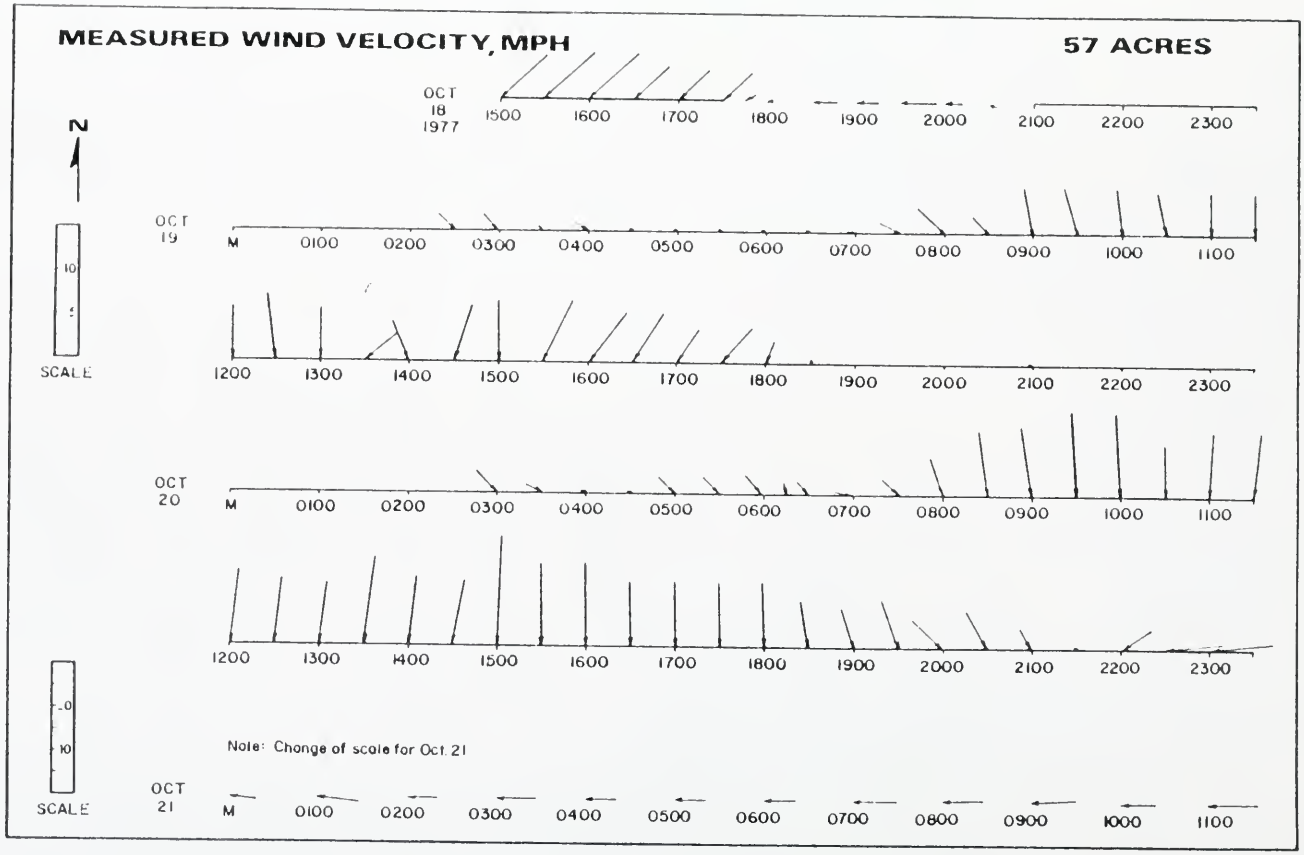

Figure B.18 - Measured Wind Velocity, 57 Acres Canal System, October 1977.

B.3.2

SECOND DISPERSION STUDY

57 ACRES

October 16-22, 1977

THREE-DIMENSIONAL MEASURED DYE CONCENTRATION PLOTS

This section contains plotted results as follows:

DATE	TIDE	TIME
771019	Low	2140
771020	High	0330
771020	Low	0950
771020	High	1610
771020	Low	2240
771021	High	0420
771021	Low	1045

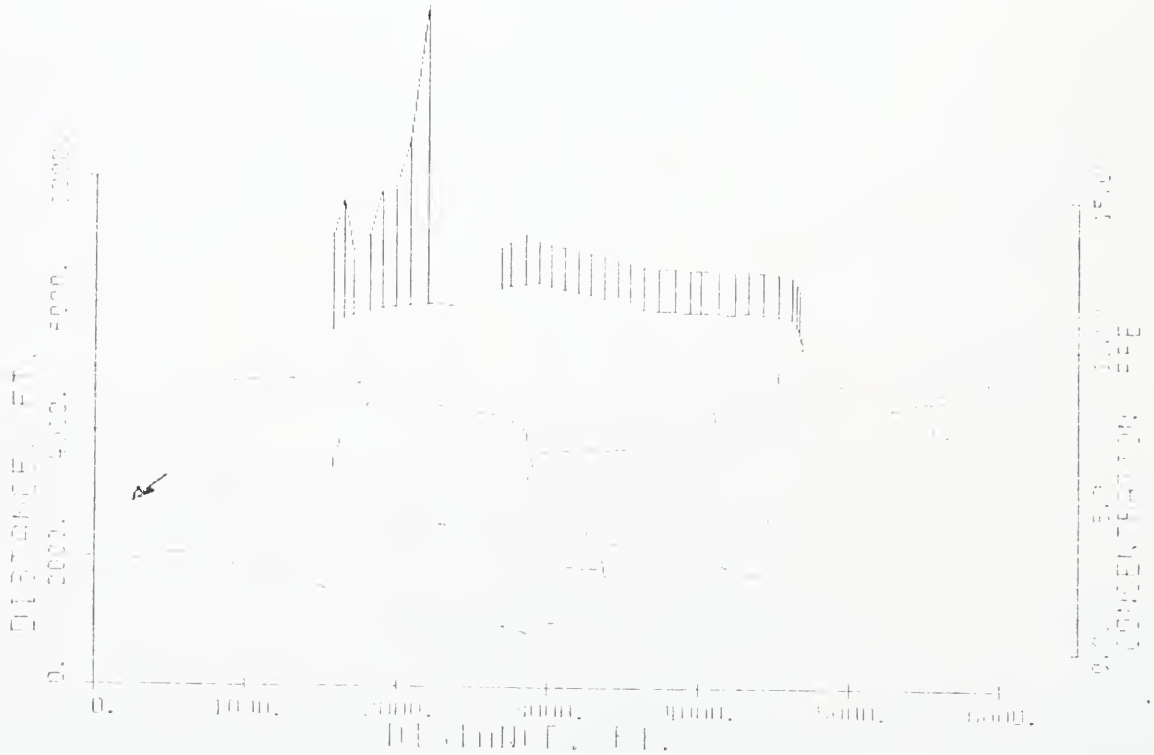

Figure B.19 - Measured Dye Concentration at 3 ft Depth Along Centerline 57 Acres Canal System, Palm Beach County, Florida Date: 771010. Tide: Low. Time: 2140. Peak: 10.00 ppb.

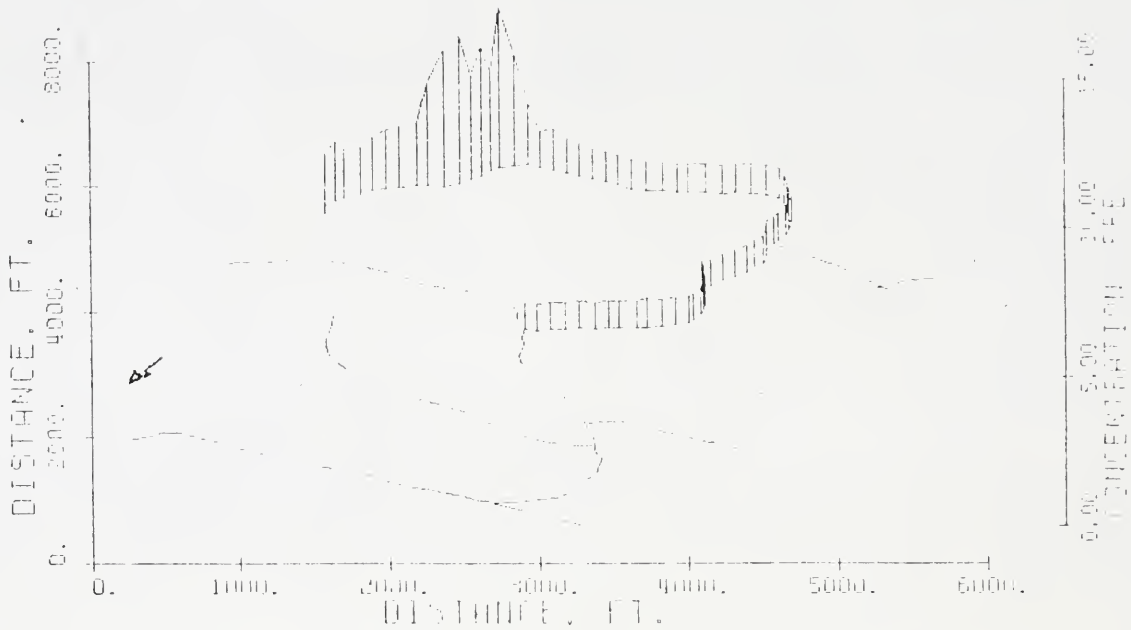

Figure B.20 - Measured Dye Concentration at 3 ft Depth Along Centerline 57 Acres Canal System, Palm Beach County, Florida Date: 771020. Tide: High. Time: 0330. Peak: 5.40 ppb. Azimuth = 210 Degrees Elevation = 25 Degrees

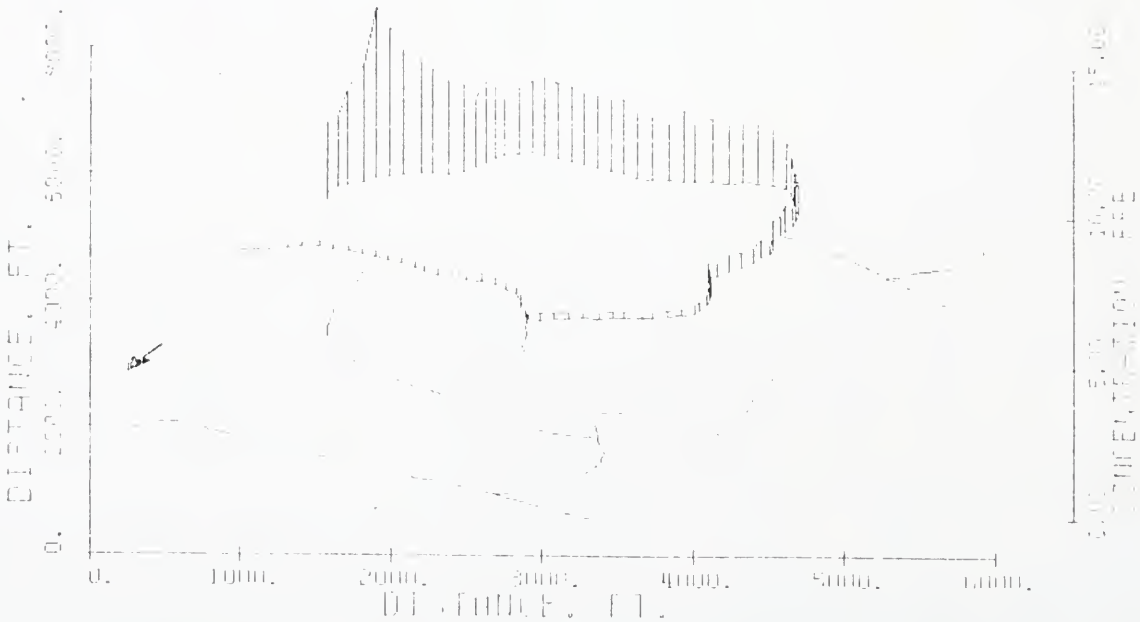

Figure B.21 - Measured Dye Concentration at 3 ft Depth Along Centerline 57 Acres Canal System, Palm Beach County, Florida Date: 771020. Tide: Low. Time: 0950. Peak: 5.70 ppb. Azimuth = 210 Degrees Elevation = 25 Degrees

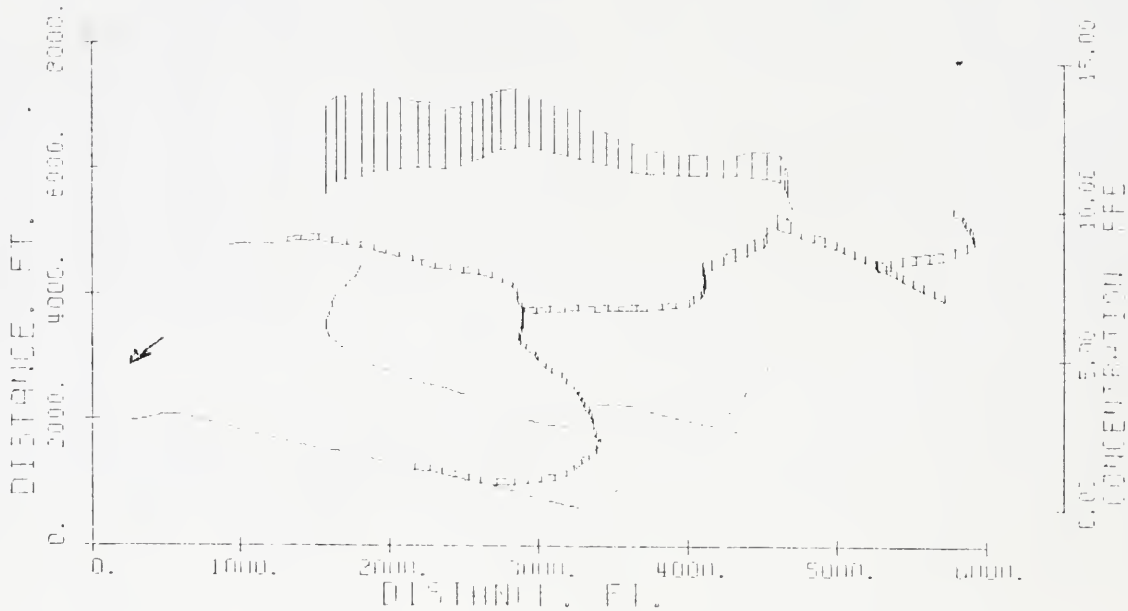

Figure B.22 - Measured Dye Concentration at 3 ft Depth Along Centerline 57 Acres Canal System, Palm Beach County, Florida Date: 771020. Tide: High. Time: 1610. Peak: 2.90 ppb. Azimuth = 210 Degrees Elevation = 25 Degrees

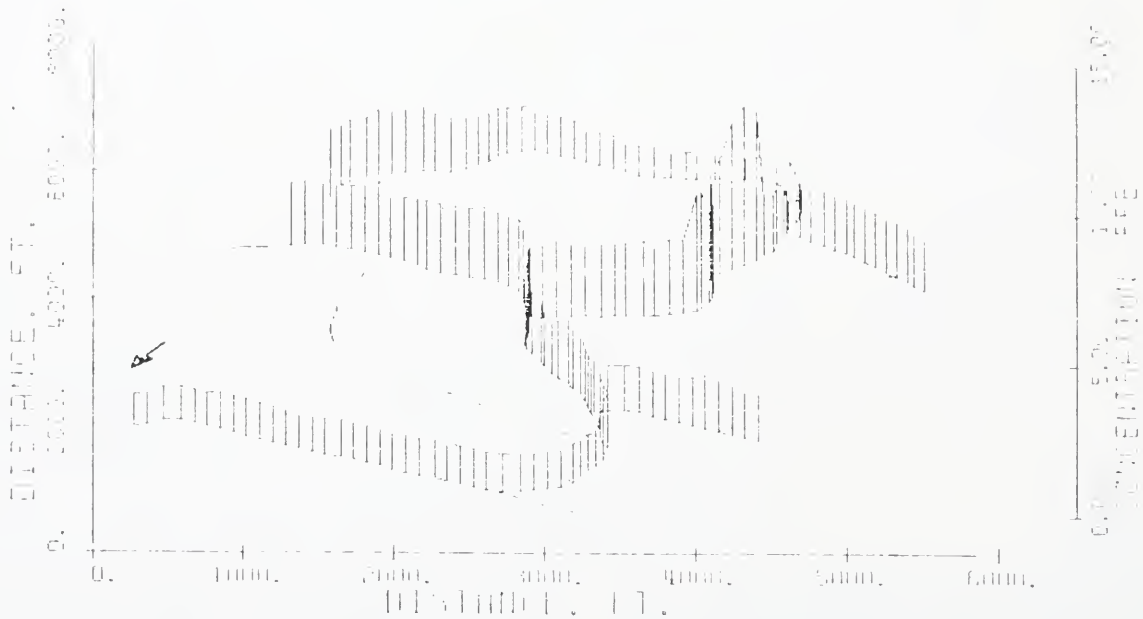

Figure B.23 - Measured Dye Concentration at 3 ft Depth Along Centerline 57 Acres Canal System, Palm Beach County, Florida Date: 771020. Tide: Low. Time: 2240, Peak: 5.30 ppb. Azimuth = 210 Degrees Elevation = 25 Degrees

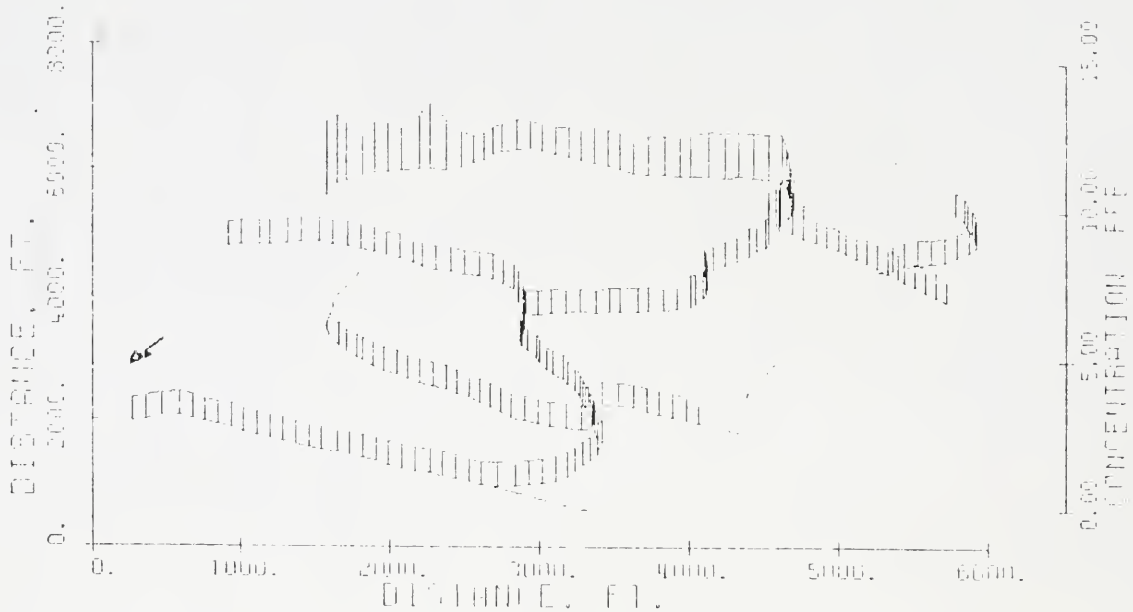

Figure B.24 - Measured Dye Concentration at 3 ft Depth Along Centerline 57 Acres Canal System, Palm Beach County, Florida Date: 771021. Tide: High. Time: 0420. Peak: 2.20 ppb. Azimuth = 210 Degrees Elevation = 25 Degrees

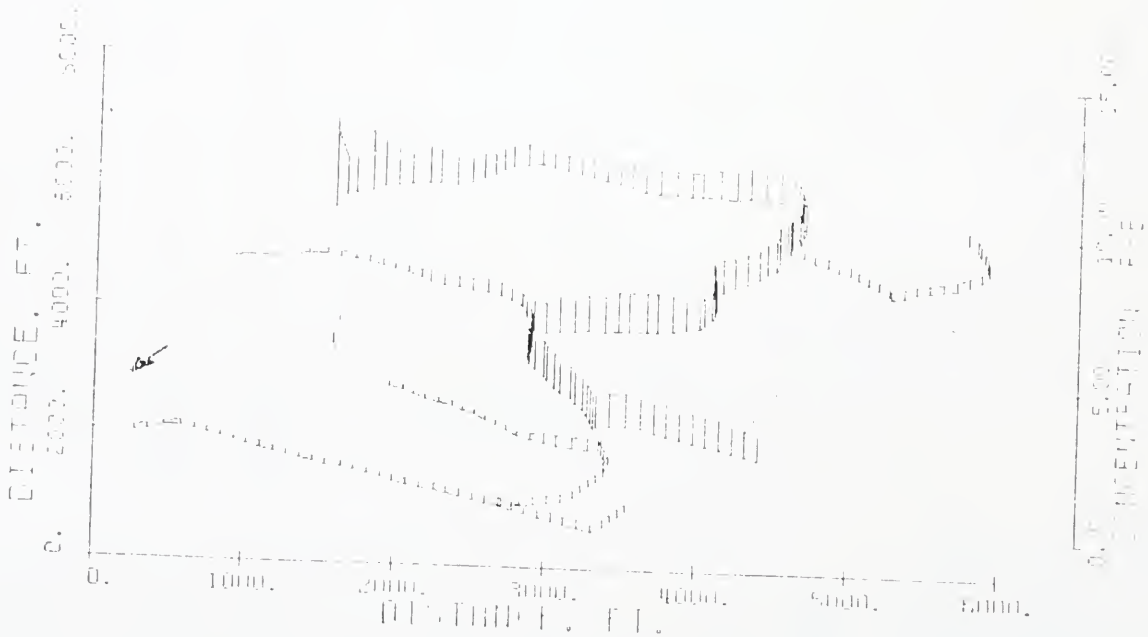

Figure B.25 - Measured Dye Concentration at 3 ft Depth Along Centerline 57 Acres Canal System, Palm Beach County, Florida Date: 771021. Tide: Low. Time: 1045. Peak: 3.00 ppb. Azimuth = 210 Degrees Elevation: 25 Degrees

B.3.3

SECOND DISPERSION STUDY

TWO-DIMENSIONAL MEASURED DYE CONCENTRATION PLOTS

This section contains plotted results as follows:

REACH	TIDE
AD	Low
AD	High

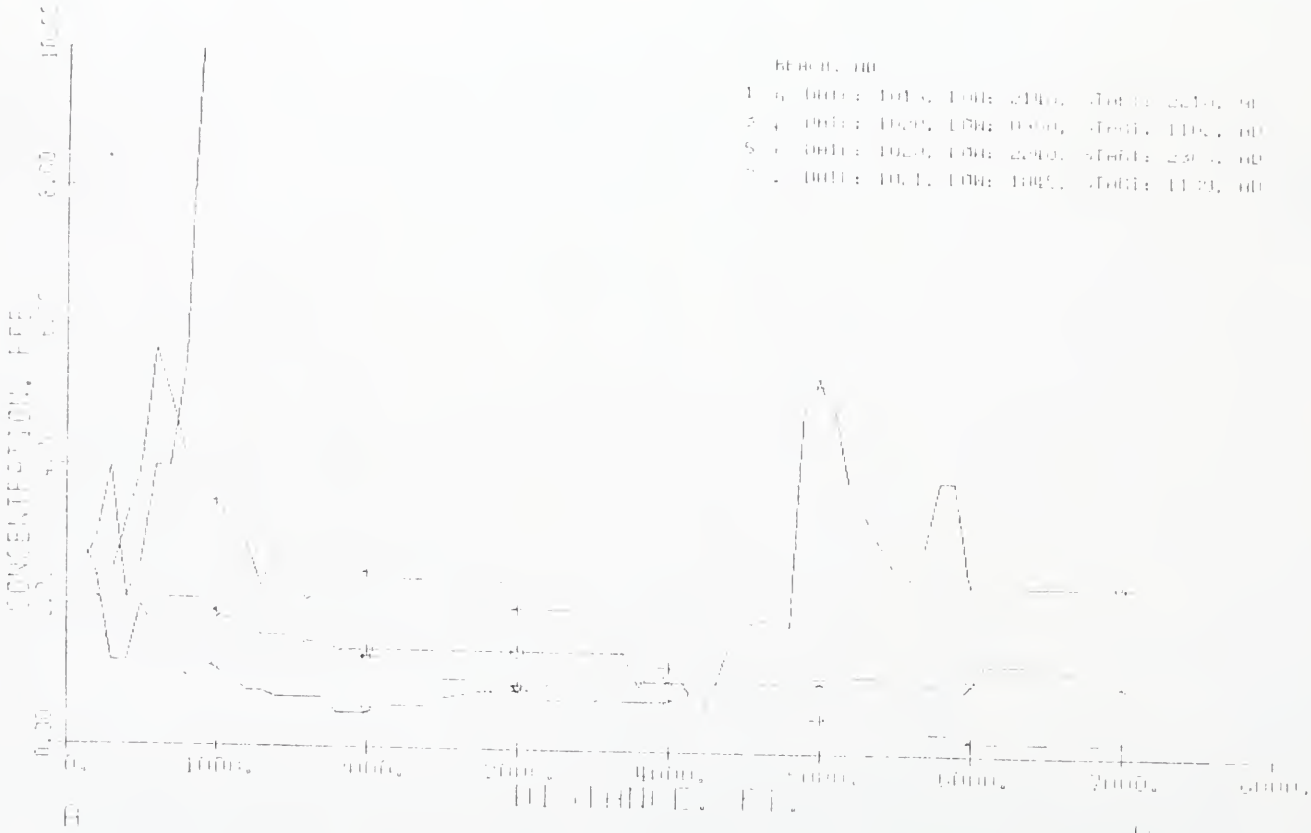

Figure B.26 - Measured Dye Concentration at 3 ft Depth Along Centerline 57 Acres Canal System, Palm Beach County, Florida, October, 1977.

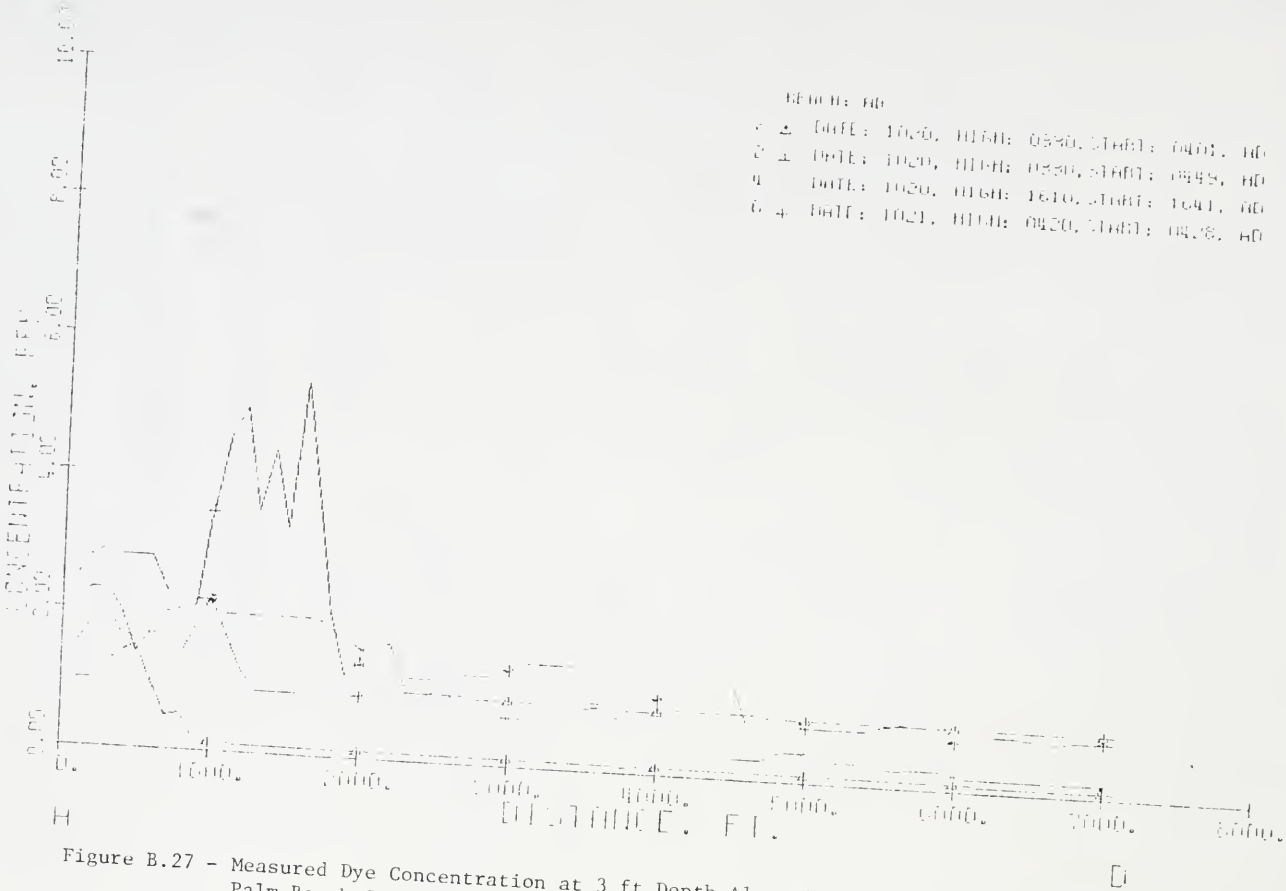

Figure B.27 - Measured Dye Concentration at 3 ft Depth Along Centerline 57 Acres Canal System, Palm Beach County, Florida, October, 1977.

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BIOGRAPHICAL SKETCH

Frederick W. Morris IV was born on December 3, 1934, in Philadelphia, Pennsylvania. He graduated from St. Paul's School, Concord, New Hampshire, in June, 1952. He was admitted to Yale University, New Haven, Connecticut, and graduated in June, 1956, with a B.S. in Industrial Administration.

Following a year as a trainee with Leeds and Northrup Company, an electronic instrument manufacturer in Philadelphia, he joined the U.S. Navy and served as an Air Controller and Navigator with Airborne Early Warning Squadron VW-2.

In 1961 Mr. Morris rejoined Leeds and Northrup Company, where he worked as an Applications Specialist in telemetry and supervisory control systems. Until 1968 he attended night school, and earned an M.S. in Electrical Engineering from the University of Pennsylvania. He then enrolled at the University of Miami, Miami, Florida, completed the course requirements for a masters degree, and joined the Reentry and Environmental Systems Division of General Electric Company in Philadelphia as a Water Resources Engineer. In August, 1973, he completed his thesis for the University of Miami and graduated with an M.S. in Ocean Engineering. He joined the Hydraulic Laboratory in the Civil Engineering Department at the University of Florida in 1974 as a Graduate Research Assistant, and was promoted to Associate level in 1976.

Mr. Morris is married to the former Susan Schuck, of Claymont, Delaware, and has three children, Holly, Fred, and Abigail.

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

B. A. Christensen
Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

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Associate Professor of Environmental Engineering Sciences

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

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