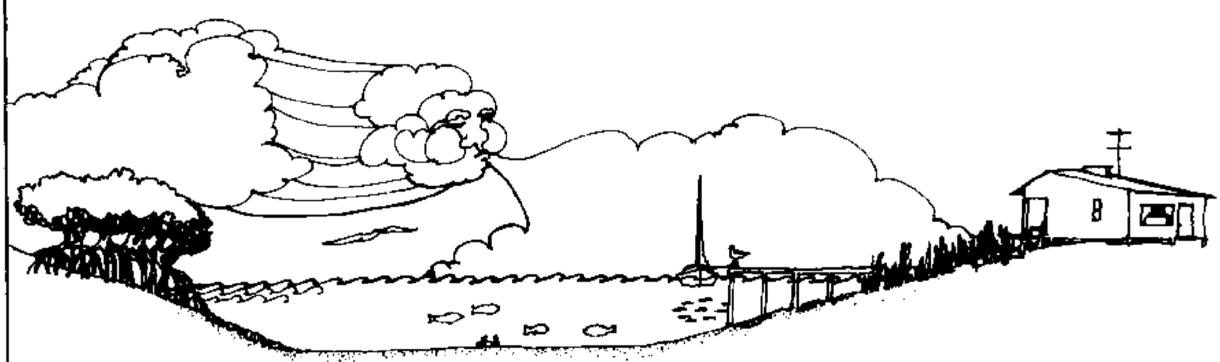
Hydrodynamic Factors Involved in Finger Canal and Borrow Lake Flushing In Florida's Coastal Zone

Volume I of II



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Department of Civil Engineering
University of Florida
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Chapters 1-15

HYDRODYNAMIC FACTORS INVOLVED IN FINGER CANAL AND BORROW LAKE FLUSHING IN FLORIDA'S COASTAL ZONE

by

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NOTATION

a - tidal amplitude, (L)

a_o - constant of integration

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

- drainage area, (L^2)

A_h - Shield's entrainment function, (dimensionless)

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 (4.31)

b - bottom width, (L)

B - top width, (L)

$\bar{B}(z)$ - function defined by Equation (4.29)

c_A - cross-sectionally averaged concentration, (dimensionless)

c_I - concentration of lateral inflow, (dimensionless)

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

c_m - concentration of rectangular distribution (Section 3.6), (dimensionless)

$\bar{c}$ - turbulent time mean concentration, (dimensionless)

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

c_o - initial concentration, (dimensionless)

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

- runoff coefficient, (dimensionless)

d - depth, (L)

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

d_o - mean tidal depth, (L)

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

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

e - exponential constant = 2.718

e_{ijk} - error in cell ijk

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_n - numerical 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

- function of ( )

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

F_m - dimensionless location of center of mass in cell (Section 3.6)

$F_1(\eta)$ - function defined by Equation (4.38)

$F_2(\eta)$ - smooth bed form of $F_4(\eta)$

$F_4(\eta)$ - function defined by Equation (4.39)

$\bar{F}_1(k)$ - 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 3)

- number of segment

I - rainfall intensity, (L/T)

j - number of reach (Chapter 3)

- number of lateral layer

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

- constant of proportionality (Equation (5.7))

- number of vertical layer

K - dimensionless dispersion coefficient (Chapter 3)

- decay coefficient, (1/T)

K_o - constant associated with an initial value, (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 (Section 4.1), (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 - coefficient for power form of uniform flow equation for mean velocity in the rough, turbulent (M) range, $(L^{1/3}/T)$

- mass of pollutant released, (M)

n - Manning's coefficient (dimensionless)

N - number of tidal cycles

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

- 6.02252×10^{23} mol^{-1}

N_i - number of concentration distributions in cell (Section 3.6)

N_u - number of upstream reaches (Eq. (3.9))

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

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

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

p - variable used in Section 4.4

- permissible deviation from background velocity (Equation 4.51)

- number of straight lines

P_n - point on the logarithmic velocity profile

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

- rainfall depth, (L)

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)

$\bar{Q}$ - discharge defined by Equation (6.3), (L^3/T)

r - radius of bend, (L)

- ratio of equivalent sand roughness to effective grain size (dimensionless)

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

R - hydraulic radius, (L)

R_i - Richardson number, (dimensionless)

R_m - dimensionless width of distribution in cell (Section 3.6)

s - inverse side slope, (dimensionless)

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

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

S_b - bed slope, (dimensionless)

S_f - slope of energy grade line, (dimensionless)

t - time, (T)

- temperature, (°C)

t' - time since low tide, (T)

T - tidal period, (T)

T_f - flushing time, (T)

T_o - duration of rainfall excess, (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_o - cross-sectional mean velocity at upstream section of reach, (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_1 - constant defined by Equation (4.54)

u_2 - constant defined by Equation (4.56)

u_3 - constant defined by Equation (4.59)

u_4 - constant defined by Equation (4.64)

v - lateral velocity component, (L/T)

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

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

w - vertical velocity component, (L/T)

w_s - wind speed, (L/T)

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

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

x' - distance from tidal entrance, (L)

X - distance, (L)

y - lateral coordinate direction

- depth, (Chapter 13), (L)

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

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)

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

∂ - partial derivative operator

Δ - dimensionless distance traveled by characteristic velocity to reach node (Equation (3.34))

ΔA - increase in cross-sectional area, (L^2)

$\Delta\tilde{F}(\ )$ - truncation terms of numerical approximation

ΔH - head loss, (L)

ΔQ - change in discharge along length of reach, (L^3/T)

Δt - time increment, (T)

ΔV - increase in volume, (L^3)

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)

ε - small number (Equation 7.11)

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

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

- bank slope angle, (degrees)

κ - von Karman's constant = 0.4

υ - kinematic viscosity, (L^2/T)

ξ - dimensionless longitudinal variable

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_{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)

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

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

ϕ - angle between interface and positive x-direction, (degrees)

- angle of repose of cohesionless material, (degress)

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

ω - tidal frequency, (1/T)

Subscripts

av - average over two time layers

b - bottom layer

c - center of cell (Section 3.6)

f - variables in freshwater layer above saltwater wedge

ℓ - vertical layer

\- longitudinal direction

LT - low tide

m - spatial mean value

max - maximum value

p - previous time level (Section 3.6)

s - variables in saltwater wedge

t - tidal variables

TE - tidal entrance

w - wind variables

0 - node at upstream section of reach

1 - node adjacent to upstream node of reach

2 - node, two away from upstream node of reach

Superscripts

n - time level

$\bar{}$ - turbulent time mean value

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

$'$ - d/dX

CHAPTER 1

INTRODUCTION

1.1 Background Information

Florida's population has been growing at a rate of about twice that of the world and three times that of the United States. This rapid growth 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 for the natural conditions at the site. Large canal projects constructed in ecologically sensitive areas have destroyed aquatic nurseries, 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. This report will show how canal networks can be designed to work with nature and to fit into the coast without adverse effect on the environment.

The story of residential canal development in Florida begins early in this century, when the state was still considered as 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 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, and even sometimes as a wasteland, which had to be conquered and transformed to "useful" property. This perceived need became embodied in 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 those early years of statehood the development of remote areas and wetlands was made easier by early surveys and maps, and was encouraged by the federal and state governments through disposition of federal and state lands by direct sales and grants to homesteaders.

In 1896 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, L., 1974, p. 69], Richard J. Bolles, introduced the contract method of selling reclaimed (by the state), subdivided, and unimproved land on an installment basis. This enterprise, which resulted in 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 only do so 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 [Redford, P., 1970, in Carter, L., 1974, p. 75] were filled with six million cubic yards of the bay bottom. While this was not a canal development, the dredge and fill technique had been 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 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 area until the 1960's.

In 1962 draglines, bulldozers, and a hugh tree-crusher began extensive alteration of a site near the southwest coast of Florida. By 1974 an irregular area had been leveled for a distance of twenty-five miles north and south across the Big Cypress 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, L., 1974, pp. 236-240].

A study of the area by the EPA [Carter, M.R., et al, 1973] showed that dredging has drastically increased runoff to the canals, thereby decreasing the area of potential inundation during the wet season with subsequent undesirable ecological site effects [Carter, M. R., et al, 1973, p. II-3] and interrupting sheet flow over a wide area. The canals have 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 [Carter, M. R., et al, 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, L., 1974, p. 29]. The sudden awakening of citizen consciousness to the environmental stress brought on by the exploitive style of development, and the subsequent revival 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 reasonably 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 [as given by Florida Trend Magazine, June, 1974, in Carter, L., 1974, pp. 32-33]. This shows the extensive acreage involved and the current and ultimate population figures for which the developers are 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, L., 1974, po. 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 are those constructed below mean low tide by dredging and filling shallow bay bottoms (Figure 1.1).
2. *intertidal* developments 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, estuaries, lakes, or other wetlands (Figure 1.2).
3. *inland or upland* canals 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 makes the layout of housing lots and access roads most convenient, the fill being used to elevate the land surface to meet state criteria for hurricane tide and flood protection. In the process of construction, dredges 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 higher elevation, upland Miami oolite and Key Largo limestone substrates would first be cut with narrow, parallel, vertical ditches to a depth of perhaps ten to fifteen feet. Then the area between the ditches would be blasted and dredged into long, straight, vertical-walled channels. In the process of dredging, the bottoms of the channels were overturned and clouds of silt were carried out to nearby tidal waters, where they were deposited to smother large areas of bottom life. 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 possibly pollutants into the aquifer. Examples are cited in Griffin [1977, Appendix A, in Morris et al, March, 1977, 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 in many canal investigations 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 have limited navigability as well. Writing about residential canal systems in the Florida Keys, Chesher [1974, p. 2] 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.

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 [Environmental Protection Agency, May 1975, p. 11; Walton, G.F., 1976, p. 142] show such a trend, which seems to indicate that this is not a local condition. Thus the Environmental Protection Agency (EPA) recommended that "an appropriate canal depth for shallow draft pleasure craft should be no more than four to six feet below mean low water", based on measurements of vertical dissolved oxygen (DO) profiles and numerical flushing models [EPA, May 1975, 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 acceptance 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 a distance underground 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 interface depth below ocean sea level was derived under simplifying 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 less in amplitude. The typical tidal energy flux into a typical Florida canal 80 ft wide 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, May 1975, 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 [1977, in Morris, et al, March, 1977, 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-sections, and is a factor in the analysis of inlet stability. Velocities which are too low are accompanied by desposition 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 pollution are stormwater runoff, septic tanks, sewage treatment plant effluent and boats and houseboats. The contents of these pollutants vary widely and have been the subject of numerous studies.

Stormwater runoff contains materials which collect on streets, roofs, and lawns and are channeled in drains, storm sewers, and drainage ditches. They include chemicals such as insecticides, herbicides, and fertilizers; animal wastes and sewage; oil and grease; chemical products from cleaning operations; garbage, refuse and trash; and dead or dying vegetation washed into the canals 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 constituents 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, & Associates, 1970, in Barada and Partigan, 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, May, 1977, 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 as well. Regulations in particular areas may require pump-out facilities for household wastes, since pollution from petroleum products cannot be effectively controlled by relying only on canal flushing.

In a study of fifty canal systems in the Florida Keys, of which forty-four were man-made canals, Chesher [1974, p. 12] 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.

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 site 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 in a Development of Regional Impact (DRI) report.

1.3 Present State of Canal Design

Snyder [1976] has summarized the adverse effects of canal design criteria which are established for the convenience of the developer and without regard for the environment (Figure 1.5). 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 other residential canal systems, in properly selected sites.

Present canal design is accomplished, as implied above, 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 that there be no significant adverse effect on the environment. But what tools are available to him 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. Such evaluations, based on inadequate models, can give grossly distorted predictions.

1.4 Objectives of the Research

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. To this end a number of related reports and hydrodynamic surveys have already been published [Morris, Walton, and Christensen, 1975; Walton, Morris and Christensen, 1975; Walton, Morris, Evans, and Christensen, 1975; Morris and Christensen, 1976a; Hydraulic Lab, University of Florida, March, 1976; Morris and Christensen, 1976b; Morris, Walton, and Christensen, 1977; Morris, Walton, Dlubac, and Christensen, 1977]. The canal design manual will be a summary of all the work undertaken on this project.

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 results of this canal design research project will be synthesized into a canal design manual for the practicing engineer, planners, and regulatory agencies.

This research project has been 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 at 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 they 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 needs for field data may be catagorized as follows:

1. need for data for analyzing whether an existing system is performing as expected,
2. need for 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 magnitudes 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 dye dispersion).

It is important that field surveys *not* be conducted before the data requirements have been defined. There will be a temptation to get to the site as quickly as possible and start measurements, which will almost inevitably result in some wasted time and some missing information of importance.

The field measurement portion of a given design project therefore 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. The objectives of the field measurement and data analysis portion of the canal research project, therefore, were to determine the relative significance of various canal parameters, to describe field surveys and measurement techniques, to obtain representative data for Floridian canals and to develop reliable methods for analysis of these data.

Physical modeling is essentially a research tool used for determing 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 tool in design, 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 for canal designers [Morris, Walton,and Christensen, 1977]. 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. The development of this objective is described in Chapters 3 through 9.

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, is 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 depends upon the quality of the quantitative description of a given site, the field data, and the numerical models.

1.5 Field Observations

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 Hydraulic Laboratory in support of the canal design project, and are summarized in Chapter 2.

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 obtain 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 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 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 the 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 Philosophy of Numerical Modeling

Today, there is an abundance of numerical models to predict hydrodynamics, transport phenomenon and water quality for almost every conceivable type of waterbody. A number of these models have been so often used that the results obtained from them are considered to be reliable; that is to say no obvious discontinuties appear in the solution and computer operation does not uncover faculty programming. In fact, there are so many models that some research effort is frequently spent in either cataloging them or making comparisons between them [Grimsrud, Finnemore, and Owen, 1976; Graham, 1977].

Unfortunately, however, there is some tendency to accept a model as proven and almost blindly, as it were, apply it to one's own, frequently very different, situation. Added to this, there has also been some tendency to develop oversimplified models, or else elaborate models in which complex techniques are used to approximate terms which have only a small effect on the results and which can usually be neglected when one considers the quality of the input data or of the other approximations and assumptions made.

In a recent final draft report from the Vertex Corporation, Virginia [Horowitz and Bazel, 1977], a study was made of the investigative and legislative procedures used in the development of what the U. S. Environmental Protection Agency considered to be the six best examples of Advance Wastewater Treatment (AWT) plants in the United States. Their report not only criticized the legal aspects as embedded in the Federal Water Pollution Control Act Ammendments of 1972 and the local laws of many states, but also the data sampling procedures and the numerical analysis procedures,

> WATER-QUALITY SURVEYS are generally suspect on technical grounds, beset with irregularities in sampling and analysis, and naive in matters pertaining to hydraulics, sediments, and water chemistry.
>
> MATHEMATICAL MODELS are oversimplified and filled with elaborate guesswork. They are intricate, abstruse fictions. They rarely account for the principal feature of the waterways they claim to represent, and they are usually built from inadequate data on hydrodynamics and water quality.

Now, this view may be somewhat overstated, but it seems that many people are not aware of the old adage G. I. G. O. - garbage in, garbage out, or as it could be modified, garbage in initially, always garbage out. This statement can be looked at from another angle. Snyder [1977] examined the potential accuracy of a certain water quality model. He assumed that if each constant to be input was known to be within 95 percent confidence limits, then the results from the model, reflecting the combined additive and multiplicative effects of errors in the constants would be accurate to within less than 1 percent confidence limits - not a very reassuring prospect!

The problem may be, for example, as it certainly is in some models, that the water quality module is well defined but that the hydrodynamic module is oversimplified, or else too elaborate to be physically meaningful. A frequent oversimplification is to assume that the hydrodynamics can be reproduced with a one-dimensional model. This is fatal when used for the analysis of coastal finger canal networks in Florida because they are low energy systems in which external influences such as the wind and salinity gradients give rise to multi-layered flows with flow reversals. Similarly the elaborate hydrodynamic models tend to concentrate on accurate solutions of a complex set of equations, the complexity of which becomes redundant when one looks at how such models handle averaging processes over confined reaches with highly variable roughness and input data - usually not too well.

The aim of this research is to develop a predictive three-dimensional model which incorporates the physically important factors in canal hydrodynamics and transport. Simple closed form solutions for the velocity field have been used instead of the full governing equations by simplifying the latter equations based on available field data and by considering the relative importance of terms when compared with the probable affect of unknowns, accuracy of input data and discretization techniques.

1.7 The Canal Design Manual and Support Dissertations

It is widely recognized that information and techniques developed in a program of research usually cannot be directly applied by practicing engineers without some additional development. In particular, information and data requirements have to be evaluated after the contribution of each to the overall project has been put into perspective. Field measurement techniques, often using sophisticated research instrumentation, must be reevaluated and possible substitutes for this instrumentation found, and results of data analyses need to be generalized. In addition, input and output features of computer models must be organized so that they can be easily applied to other cases within their range of applicability by persons not familiar with the details of operation of the models.

The objective of the canal research project is to develop a rational procedure for the design and evaluation of tidal residential canal systems to assure compatibility with the environment. In addition to the development of the applications information listed above, the canal design manual will include descriptions and explanations of the fundamental physical and biological processes operating in Floridian canals, design objectives, and an example of the design process. A tentative outline for the design manual, which will be written from the information in this report, is given in Figure 1.6. This outline may have to be modified in its arrangement but not in its essential content, as the manual is written.

Two dissertations are being completed in connection with this project. One describes the development of numerical mass transport models, analysis of field data, and verification of the models using these data [Walton, 1978]. The second describes the field measurements, data analysis, the development of methods for using the numerical models in the objective design of canal systems, and the development of information for use in the canal manual [Morris, 1978].

1.8 Organization of This Report

The principal features of typical tidal canal systems in Florida are summarized in Chapter 2. This section considers the various types of marine ecosystems, geology, climatic conditions, physical 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.

The results of the field work guided the development of the basic features of the canal design model. Therefore, Chapter 2 also contains a description of the field experiments and the field instrumentation used by the Hydraulic Laboratory, an explanation of dispersion coefficients, and an interpretation of the data obtained from salinity, wind, velocity and, dye dispersion measurements. Appendices A and B contain summaries of field studies at two different sites, and Appendix C describes computer programs used in developing data presentations.

Florida's canal networks are low energy systems, that is low velocities are commonly measured, and the energy available for mixing processes is small. Because the diffusion and dispersion coefficients associated with the mixing processes are small, the numerical dispersion produced by the discretization techniques used in many models of the convection and time derivative terms of the mass-transport equation frequently dominates the natural dispersion being modeled. The result is that concentration profiles can be underestimated unless extremely small time and spatial increments, which is computationally and economically unrealistic, are used.

The accuracy and stability of various types of models and numerical techniques are examined in Chapter 3 to determine their applicability in modeling the hydrodynamics of Florida's coastal finger canal systems. A one-dimensional mass transport model is developed, using the assumption that the water surface in the system is horizontal to derive closed form expressions for the velocity field and dispersion coefficients. The most favorable scheme was chosen as a basis for extending the model to three-dimensions.

The velocity field in coastal finger canal networks is rarely one-dimensional. In fact observations often show multi-layered systems with flow reversals occurring due to wind circulation, density gradients of salinity variations, and secondary currents. However, approximate solutions of the full governing equations of momentum and continuity are unrealistic because of the averaging procedures used in determining roughness effects, wind circulation, and the effects of density gradients. In Chapter 4, the governing equations are examined and simplifying assumptions made consistent with the expected accuracy of input data and model results. Closed form solutions of the time and spatial variables for the various component parts of the hydrodynamics of a canal network are derived, which may be superimposed to determine the velocity field.

The three-dimensional mass transport equation, developed in Chapter 5, describes the transport of a substance in a canal network under the influence of the velocity field and diffusion processes. Knowing the velocity field as closed form functions of the time increment and spatial variables, the diffusion coefficients are developed from classical relationships giving similar functions. The diffusion coefficients are modified, however, to take into account the mixing that still occurs during periods of low tide when the velocities are zero.

Using the form of the hydrodynamic and transport processes discussed in the preceeding two chapters, a predictive three-dimensional mass-transport model is developed in Chapter 6 using the method of second moments. The model forms of the boundary conditions at solid boundaries, tidal entrances and of the lateral inflows are presented and are shown to be realistic from a comparison with the one-dimensional mass transport model and a simple two-dimensional finite element model of flow in the receiving waterbody.

In Chapter 7, the stability criteria for the model are developed and analysed using a series of test runs which vary the time and spatial increments. The model is transportive, conservative and is shown to be stable and accurate. In calibrating the model, field data, obtained on a canal network in southeast Florida, were reproduced numerically by varying the model parameters. The model was then run to determine its accuracy by comparing the results with a second set of data in the same canal network. These comparisons are presented in Chapter 8. A summary and conclusions are given in Chapter 9.

Appendix D is a users' manual for the model. A flow chart of the program is given in Appendix E, and Appendix F is a program listing.

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 which, unfortunately, have become widely accepted in practice through continued use. In Chapter 10 these factors are outlined and discussed as they pertain to present day design objectives and limitations. Some traditional and new design approaches are summarized and, in addition, common sources of pollution are characterized and quantified as available from the literature.

Chapter 11 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 which 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 12 distinguishes between fixed site characteristics and alterable site characteristics, and describes sources of published information on regional data which are applicable to this problem. A discussion of methods for evaluating and using these data, and subsequent field data for verification of the design completes this chapter.

In Chapter 13 an overall design process is described. This process begins with the development of quantitative design constraints and criteria, giving specific examples of each. Design elements are described, and some general guidelines as to their use in synthesizing canal networks are tabulated. The development of a trial canal design is described in terms of some of the major design factors, including the application of storm hydrographs, the determination of friction effect from a measured velocity profile, the evaluation of the potential for deposition and erosion of canal beds and banks, and stable cross-section design. The final part of Chapter 13 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 14. 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 15 is a summary of the project, together with conclusions and recommendations.

Figure 1.1 - Example of Bayfill Development in Florida.

Figure 1.2 - Example of Intertidal Development in Florida.

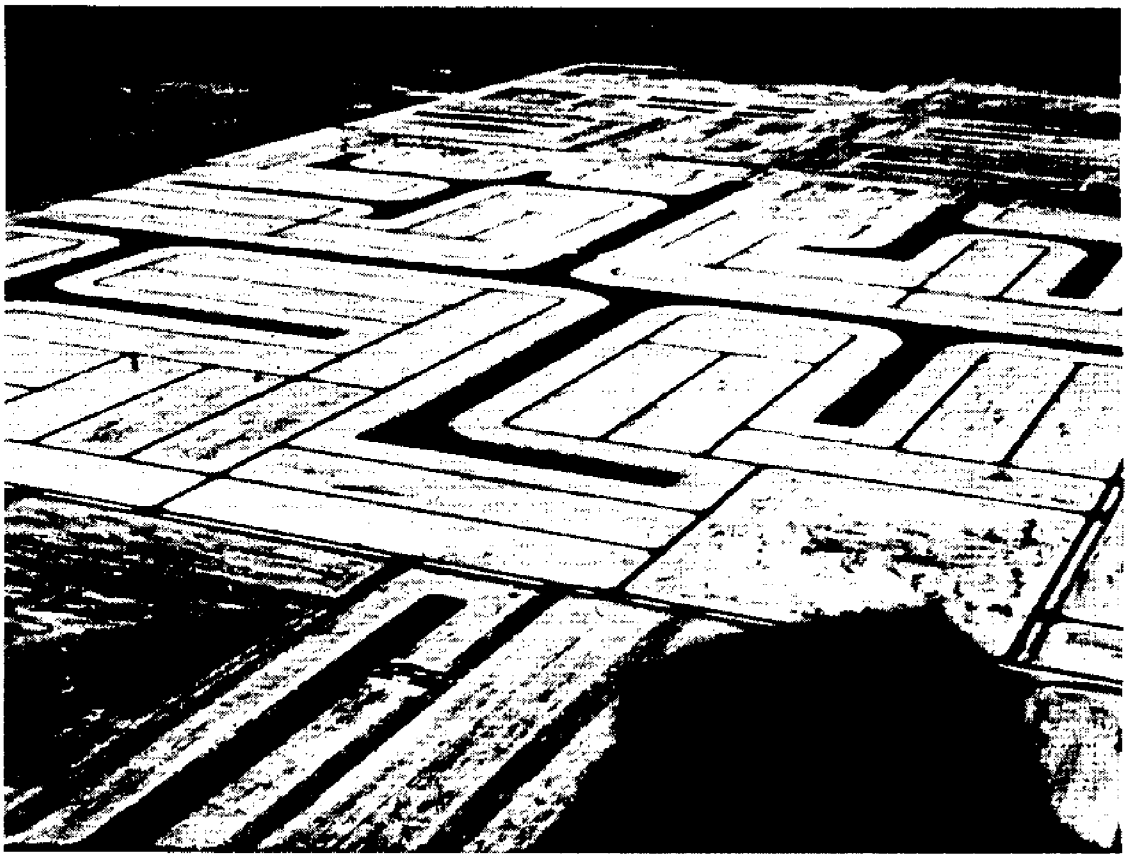
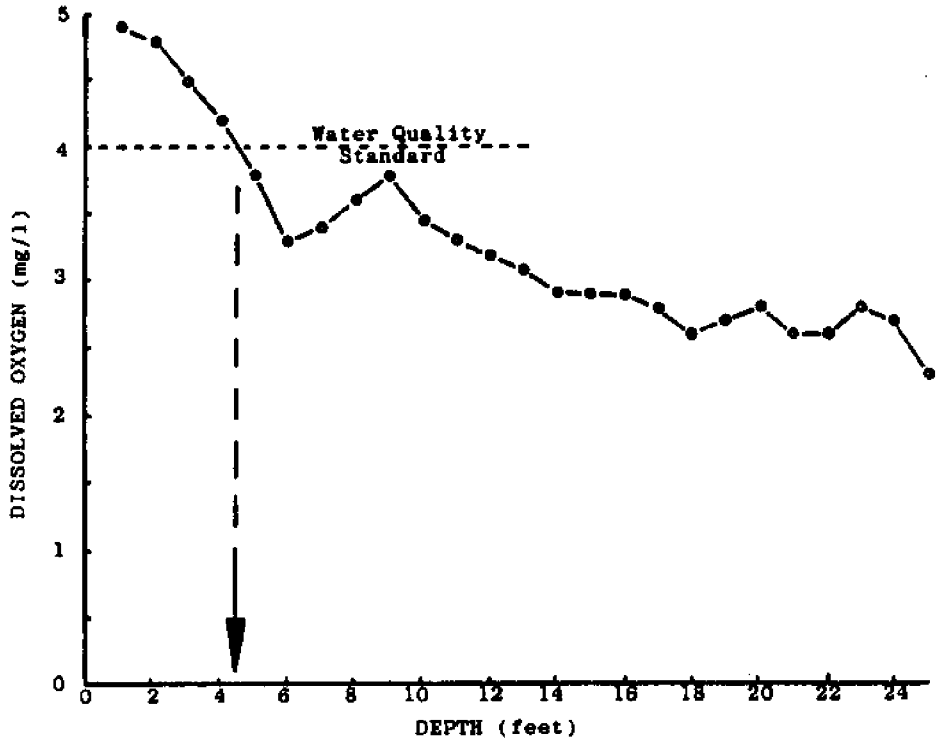


Figure 1.3 - Example of Inland Canal Development in Florida.



DISSOLVE OXYGEN STATISTICS
CANAL SYSTEMS AT FLORIDA
AUGUST 1974

Depth	No. of Observations	Avg DO	Range	% > 4.0	% ≤ 3.9	95% Confidence	Interval	Std Deviation
1	134	4.9	8.7-1.4	84	16	2.7	7.1	1.1
2	134	4.8	7.8-1.8	80	20	2.6	7.0	1.1
3	134	4.5	8.8-1.4	71	29	2.2	6.8	1.1
4	134	4.2	8.9-0.8	64	36	1.6	6.8	1.3
5	134	3.8	6.8- 0	55	45	0.3	7.3	1.8
6	134	3.3	7.0- 0	47	53	0	7.4	2.1
7	125	3.4	7.0- 0	46	54	0	7.5	2.1
8	89	3.6	7.2- 0	44	56	0	7.5	2.0
9	67	3.8	8.0- 0	46	54	0	7.4	1.9
10	40	3.4	7.1- 0	40	60	0.4	6.4	1.5
11	33	3.3	6.2-1.0	33	67	1.1	5.5	1.0
12	30	3.2	5.1-1.0	23	77	1.1	5.3	1.0
13	28	3.1	6.0-1.1	14	86	1.1	5.1	1.0
14	24	2.9	4.3-1.1	8	92	0.9	4.8	1.0
15	24	2.9	4.3-1.1	8	92	1.0	4.8	0.9
16	22	2.9	4.3-0.8	9	91	0.9	4.9	1.0
17	17	2.8	4.0-0.7	6	94	0.7	4.9	1.0
18	15	2.6	4.0-0.7	7	93	0.3	4.7	1.0
19	14	2.7	3.8-1.2	0	100	1.0	4.4	0.8
20	11	2.8	3.6-0.8	0	100	1.2	4.4	0.7
21	9	2.6	3.6-0.8	0	100	0.6	4.6	0.9
22	5	2.6	3.8-1.3	0	100	0.9	4.3	0.9
23	6	2.8	3.4-2.3	0	100	1.8	3.8	0.4
24	2	2.7	2.9-2.4	0	100	-	-	-
25	1	2.3	2.3	0	100	-	-	-

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

GOAL	INCIDENTAL FACTORS	ENGINEERING CRITERIA	RESULTING DEVELOPMENT	ENVIRONMENTAL IMPACT
	NO CENTRAL SEWAGE TREATMENT		SEPTIC TANK SEEPAGE	DOMESTIC WASTE POLLUTION
RESIDENTIAL UTILIZATION OF WATERFRONT		NAVIGABLE DEPTHS TO SHORELINE	ELIMINATION OF SHALLOWS	NO NATURAL WATER TREATMENT
		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 1.5 - Criteria for Canal Design Without Consideration of the Environment (after Snyder, R.M., *Residential Canals and Environment*, Snyder Oceanography Services, Jupiter, Florida, 1976).

CANAL DESIGN AND EVALUATION MANUAL

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CANAL DESIGN AND EVALUATION MANUAL

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Figure 1.6 - Canal Design and Evaluation Manual, Tentative List of Content.

CANAL DESIGN AND EVALUATION MANUAL

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Figure 1.6 - continued.

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	61.000	100.000	300.000	16.371
Golden Gate Estates	Collier	GAC Corporation	112.000b	0	0	
River Ranch Shores	Osceola	GAC Corporation	6.427	15,000	40,000	
River Ranch Acres	Osceola	GAC Corporation	49.356b	0	0	
Ramuda 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	4.730	12.000	30.000	
Poinciana	Osceola/Polk	GAC Corporation	47.300	70.000	250.000	0
Golden Gate	Collier	GAC Corporation	2.500	6.000	26.000	
Port Charlotte	Charlotte/Sarasota	General Development	100,640	190.677	500,000e	26,500
Port LaBelle	Hendry/Glades	General Development	31.530	50.000	130,000e	10
Port St. Lucie	St. Lucie	General Development	48.480	83.100	220.000e	9,500
Port Malabar	Brevard	General Development	43.500	70.449	185.000e	8.750
Port St. John	Brevard	General Development	5,586	9,597	25,000e	1,400
Vero Bch. Shores/Hghlds.	Indian River (2)	General Development	1.556	2,329	6.000e	658
Sebastian Highlands	Brevard	General Development	4.958	13.195	35.000e	650
Julington Creek	St. Johns	General Development	4.630b	0	0	
Deltona	Volusia	Deltona Corporation	17.500	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	15.300	33.000	95.600	1.085
Sunny Hills	Washington	Deltona Corporation	17.100	34.000	96.500	175
St. Augustine Shores	St. Johns	Deltona Corporation	2.000	6,000	12.000	0
Pine Ridge Estates	Citrus	Deltona Corporation	9.584	5.000	14.500	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	13.900	38.200	1.066
Royal Highlands	Hernando	Royal P. B. Colony, Inc.	8.000	19.000	66.000	
Holley-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	Broward	Gulfstream Land & Devmt.	5.400	11,900e	33.300e	
Venice Gardens	Sarasota	Gulfstream Land & Devmt.	3.100	6.600e	19.000e	
Winter Springs	Seminole (PUD)	Gulfstream Land & Devmt.	3.500	7.700e	21.600e	
"Argyle Forests"	Duval/Clay	Gulfstream Land & Devmt.	8 000	17.600e	49.300e	
Coral Springs	Broward	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	18.800	50.000	
Perdido Bay C. C. Estates	Escambia	Cavanaugh Comm. Corp.	2.900	15.805	36,232	365
Rotonda West	Charlotte	Cavanaugh Comm. Corp.	26.000	65.688	164,000	520
Orangewood	Osceola	Florida Land Company c	4.500	17.957	48,000	100
Winter Springs (N. Orlando)	Seminole (PUD. 560 a.)	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	2.753	7,010	17,525	500
San Carlos Estates	Lee	American Intl. Land Corp.	1.160+b	0	0	
San Carlos "West 600"	Lee	American Intl. Land Corp.	720+b	0	0	
Peace River "West"	Desoto	American Intl. Land Corp.	3.000+b	0	0	
Peace River "East"	Desoto	American Intl. Land Corp.	2.306+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	3.580	9,650	27.000	18
Palm Coast	St. Johns/Flagler	IT&T c	100.000	250,000e	750,000	600
Crescent Estates	Polk	Florando Investment Corp.	2,821	20,000	52,500	50
Solana	Polk	Treasury Investment Corp.	2.200	15.260	35.000	0
Tomoka Springs	Volusia	Recreational Systems, Inc.	2,746	1.005	3.015	0
Placid Lakes	Highlands	Lake Placid Holding Co.	7.200	15,000	35,000	600
Wellington	Palm Beach	Breakwater Hsng. Corp.	7.400	14,600	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.	3.560b	0	0	
Rolling Hills	Marion	MRI Properties, Inc.	5.400b	0	0	
Miami Lakes	Dade	Sengra c	6.900e	25.000		
North Palm Beach	Palm Beach	(Ross Brothers) c	9.600	8,900e	25,000	10.923
Palm Beach Lakes	Palm Beach	Perini Corporation c	7.000	25.000e	70.000	
Palm Beach Gardens	Palm Beach	McArthur c	6.100	25,000e	70,000	7,620
Bay Port Colony	Hillsborough	Intervest, Inc. c	2.565	9.199	30.874	0
(Fletcher)	St. Johns	Fletcher Prop., Inc. c	5.300b	0	0	

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

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

CHAPTER 2

FEATURES OF CANALS, FIELD MEASUREMENTS, AND DATA ANALYSIS

2.1 Features of Coastal Residential Canals

This section combines a literature review with a general, mostly qualitative explanation of the function of tidal 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, as this report is not intended to be a comprehensive data review. It should also be emphasized that canals for navigation, flood control, irrigation, mosquito control, and other purposes are not specificially covered by this report, as we are addressing the problem of residential canal design. 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.

The hydrodynamics of tidal canals are governed by the combined effects of climate (primarily wind and rainfall), canal system layout, channel geometry, tidal range and period, the saltwater/freshwater balance, evaporation, water circulation (as affected by tide and winds), and certain geological features. These topics will be reviewed in this chapter to the extent that they pertain to canal design. 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 system. Since the most widely used water quality indicator is dissolved oxygen (DO), it will be shown how this parameter is affected by the hydrodynamics through mixing and dispersion.

A given canal system is part of the surrounding ecosystem, which has, in its broadest sense, 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.

2.1.1 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 general coastline, 593 miles border the Atlantic Ocean and the remaining 673 miles lie on Florida Bay and the Gulf of Mexico [Corps of Engineers, 1971, p. dl; Carter, L. J., 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 boarder salt marshes, while to the south as far as Miami the barrier island chain, which is broken by fourteen of the fifty-seven 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, steep sand strips fronting seawalls. Some of the inlets open into extensive estuaries, while others were formed by storms breaching the barrier islands and 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, starting 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 improved for navigation.

North from Anclote Key to Apalachee Bay the coast changes character completely. 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].

2.1.2 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.

Generally, 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, the proximity to the Atlantic Ocean and the Gulf of Mexico, and the 4,400 square miles of surface area comprising the 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 fifty inches per year, except the Florida Keys which average about forty inches per year. The average year can be subdivided into two "rainfall seasons" throughout all of the state but 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 within 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 80 of these storms in a year, while some experience more than 100. 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, may even occur 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 occured at all times of the year, but they do not cause extensive damage.

Tropical storms, and particular hurricanes, are another matter. A tropical storm is any storm which produces high winds (above thirty-four knots) and therefore has destructive potential. A hurricane is a tropical storm with maximum winds of sixty-four knots, or more. From 1885 through 1977, sixty-seven tropical storms and eighty-six hurricanes have entered or significantly affected Florida ["Mariner's Weather Log" to 1977]. The average number of tropical storms is 1.7 per year, with a variation of from none to five in any year. Florida has not experienced more than three consecutive years without a tropical storm, nor more than five consecutive years without a major hurricane.

The probability that hurricane force winds will impact a particular 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 approximately 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, 1972, pp. 45-70].

2.1.3 Physical Features of Floridian Canal Systems

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

Christensen [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 which 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.

2.1.3.2 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 in the euphotic zone, 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 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 which 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.

2.1.3.3 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 (Chapter 1). 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.

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 in Palm Beach County (Figures 2.7 and 2.8) revealed a lag of thirty-six minutes between the south entrance, F, and Station A at the end of the south loop, and a lag of forty-eight minutes between the north, K, or south entrance and the end of the lake spur canal, Station T (Figure 2.9). The tidal peak attenuation from Station K to T is of the order 0.1 ft, and from Station F to A (Figure 2.10) is of the order 0.04 ft for an incoming tide.

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. Attempts to measure the slope with tide guages showed it to be of the order 0.00005 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 which assumes that the water surface is always horizontal [Walton, 1978]. The elevations and velocities compared within 2 percent, which was concluded to be sufficient justification for ignoring the surface slope considering that combined measurement accuracy for tidal elevations, current velocities, dye studies, and predictions using numerical models would be significantly lower. The physical explanation which 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.

2.1.3.4 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

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

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 $\frac{\text{lb-ft}}{\text{sec}}$ to hp

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

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

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

where a and A_s are to be worked in 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 availabe for flushing on the ebb flow at any location in the canal system is given by Equation (2.2), using the mean tide level surface area upstream (in the direction away from the tidal entrance) of that location for A_s. For example, the total horsepower available in the 57 Acres canal system is 105 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, for flushing, by energy from the wind and salinity gradients.

2.1.3.5 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 (Figure 2.11) indicate that even in a straight, 2200 ft dead-end canal the flow is not at all uniform in any of the coordinate directions (Figures 2.12 and 2.13). 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 which are uncovered at low tide.

Secondary currents 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 2.14. Secondary flows are an important design element because they are instrumental in vertical mixing.

2.1.3.6 Dispersion Coefficients and Flushing Time

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 carries, or convects, pollutants with it, while concentration gradients cause the pollutants to distribute themselves in the direction which 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 and, consequently, turbulent diffusion. The combined effect of convection and turbulent diffusion in the longitudinal direction in a one-dimensional sense (e.g., in a cross-sectionally averaged numerical model) is called longitudinal dispersion. As one might expect, the magnitudes of vertical and lateral diffusion and longitudinal dispersion vary with location and time in a tidal waterbody.

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. Ignoring the dynamics of the circulation and the details of the flushing process, assuming that the flow of polluted water into the canal predominates over the tidal flow and that steady state conditions prevail (i.e., that the pollutant is removed at the same rate as it is introduced) both the flushing time, T_F, and the mean residence time, T_R, are given by

$$T_F = T_R = \frac{V}{Q_i} \tag{2.3}$$

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 (2.3) 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, B. H., 1951; Kupferman, S., 1974] complete mixing is assumed to take place successively in segments with lengths determined by the tidal excursion. While such models attempt to take variation in 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.

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 references in Chapter 3), the comments by EPA [May, 1975 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 Mangement Model (SWMM), 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 chose the Columbia River Model (CRM) version of the original Water Resources Engineers model [Callaway, 1970] for simulating flushing in canals.

The effects of diffusion and dispersion are incorporated into numerical models of pollutant concentration by means of diffusion and dispersion coefficients. Such 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, May, 1975, 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 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 dispersion, such as the model by Walton [1978], 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 schedules. Thus, for example, the effect of an unusually large pollutant point loading of limited duration can only be evaluated by a model which can simulate the circulation and dispersion over successive tidal cycles.

2.1.3.7 Bank and Bottom Materials

The magnitude of the bank and bottom roughness, and variations in the roughness with location in the canal network, control 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 procedure for each is given in Chapter 13 with development of appropriate equations.

The bottoms of the Floridian canals surveyed by the Hydraulic Laboratory consist 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 mostly made of sand or sandy soil. The stability of these banks and bottoms is dependent upon the grain size distribution and velocities in the channel. Banks and bottoms 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 of inlets, 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.

Bank and bottom 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) (Figure 2.7). 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 while the bottom sizes ranged from 0.11 to 0.30 (sand) and from .00024 to .0075 (silt). There was no significant difference between the samples from the two different locations.

2.1.4 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 2.15 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. 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 2.15.

The mean, minimum, and maximum values for each parameter, compiled from a selected set of data, is herein very 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 base. 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 means, 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 to a maximum of 40 ppt in the Frenchman's canal system (Figure 2.3). 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 [Appendix A in Morris, et al, March, 1977, 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. DO is commonly used as an indicator of water quality. 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 0 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 criteria 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 freshwater 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 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 is a sudden resuspension of bottom sediments, which can be caused by dredging or a storm. It is evident that anaerobic conditions occur naturally in natural channels, but the state regulations for dissolved oxygen in canal waters only permit lower limits 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 only result 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]. These results come from a principal component analysis, a statistical technique which identifies the parameters

which 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.

3. A canonical (linear combinations) correlation analysis indicates the amount of correlation or association between a series of linear combinations of data from two data subsets. This analysis indicated a substantial association between water quality parameters, and the physical characteristics. Specifically, it was found that,
   a. "large [surface area] canal systems tend to have higher average dissolved oxygen concentrations" [Bailey, 1977, p. 168],
   b. "increases in canal width increase average oxygen levels; but [increase 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, only conjecture is available. 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 which 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 used in Bailey's model.

2.1.5 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. 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.
2. storm and waste water removal rates. The characteristics of the surficial deposits, the surface topography and the presence or absence of an aquiclude above the aquifer, will influence the rate of infiltration through the soil and into the aquifer, as well as the rate of infiltration into canals, if significant.
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.

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 2.16 [Puri and Vernon, 1964, pp. 7-13]. The geologic structures and stratigraphy of the state are highly variable from one region to another (for example, see Figure 2.17) 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 2.18. As can be surmised from the latter figure, each site will have a unique stratigraphy which will have to be investigated in detail 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 of limestone 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 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 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 inpervious [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. Organic soils do absorb large quantities of surface water until they become saturated, so they generally only contribute to runoff during heavy rainstorms [Veri, et al, p. 19].

Marl is relatively impervious and has poor aquifer potential. Rainfall runs off 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].

2.1.6 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 and 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 entirely 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".

Salinity gradients and/or stratification commonly occur in Floridian canals during the wet (summer) season due to runoff from rainstorms. Figure 2.19c 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 2.19a) 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 2.19b) and that DO gradients (Figure 2.19d) 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.

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

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

If conditions are favorable for stratification in a particular canal the 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 entrances 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 4.

2.1.7 Terrestrial and Marine Ecosystems

The coastal ecosystems of the United States have been categorized into six major groups based on the flow of potential energy [Odum, Copeland, and McMahan, 1974]. Potential energy, as used in this classification scheme, includes such sources as the sun, tides, waves, salinity, mixing, nutrients, and pollution -- sources which provide energy to or cause a stress on one or more identifiable units within the system. The six categories of coastal ecological systems are:

1. Naturally stressed systems of wide latitudinal range.
2. Natural tropical ecosystems of high diversity.
3. Natural temperate ecosystems with seasonal programming.
4. Natural arctic ecosystems with ice stress.
5. Emerging new systems associated with man.
6. Migrating subsystems that organize areas.

Florida has examples in each category except arctic ecosystems, as shown in Figure 2.20.

The coastal ecosystems viewpoint of a coastal site is one which organizes the interrelationships of the predominant influences, whether they be physical, biological, geological, or derived from man's activities. The common denominator of these interrelationships is the amount of potential energy (measured in calories) which is used by each unit or subsystem (compartment) as it does the work necessary to maintain itself and its interaction with other units. The principal types of energy sources and products in the coastal system are

1. light energy, which drives photosynthesis of plants and thereby supports the food chains which fuel biological populations,
2. organic fuels, which serve as food sources for bacteria and higher animals, such as the organic loads of rivers, the organic detritus from swamps and mangroves, and man's waste products.
3. mechanical energy, the transfer and utilization of the fluid momentum of the tides, waves, and wind,

and its conversion to heat in the process of convection, mixing, and overcoming friction.

The flows of energy are directed in accordance with the physical, biological, chemical, and geologic structures or demands within the system, being transformed into useful work and eventually passing off as heat.

Each of the first five major categories of coastal ecological systems is divided into a number of types of subsystems [Odum, et al, 1974, p. 25]. If Floridian canal environments were to be described by such a classification system, we would commonly find combinations of the following kinds of subsystems: mangrove (B-1), tropical meadows (B-3), tropical inshore (bay) plankton (B-4), marshes (C-4), medium salinity plankton estuary (C-9), sheltered and stratified estuary (C-10), sewage waste (E-1), pesticides (E-3), dredge spoil (E-4), and piling (E-12). The characteristics of each of these types of subsystems are described, with examples, in Odum et al [1974].

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 covered here, and in Chapter 12.

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 [Chesher, 1974, p. 97].

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 improtant species, such as mangrove, turtle grass, and marsh grass, have been undertaken 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 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 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.

2.1.7 1 Lawn Grasses

Lawn grasses lower the rate of flow of stormwater and runoff, increasing the rate of uptake of nutrients in proportion to the resistance to flow. Since the direction of flow can be controlled by filter mounds and swales (Figure 2.21), 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.

In experiments conducted at the U.S. Army Waterways Experiment Station, Environmental Effects Laboratory, Carlson [1974] measured the uptake of nutrients and pollutants by a grass sod model 5 ft wide by 20 ft long over low permeability clay at a 2 percent slope. He found the natural "treatment" of "average (secondary effluent supplemented with 13.4 mg/l of nitrogen as ammonium nitrate and 0.2 mg/l each of cadmium, copper, lead, manganese, nickle and zinc)" wastewater to be as follows: [Snyder, 1976, pp. 61-62], The wastewater was generally distributed into 20, 30, and 50 percent for subflow, evapotranspiration, and runoff, respectively. Analyses showed that 100, 95, 91, and 75 percent of the ammonium, nitrate, and organic nitrogen and phosphorus, respectively were retained in the model. The balance of the nitrogen and phosphorus remained in the runoff, with essentially none in the subflow. The grass harvest removed 31 percent of the nitrogen and 6 percent of the phosphorus. The model retained from 98 to 100 percent of the cadmium, copper, manganese, and nickel: 91 percent of the lead; and 72 percent of the zinc. The balance was lost in runoff, except for 2 percent manganese and 6 percent zinc that were lost in the subflow. The grass removed 4 percent of the applied copper, lead, and nickel; 9 percent of the zinc; 13 percent of the manganese; and none of the cadmium. The elements retained in the model were strongly fixed or volatized since they were not readily extracted or exchanged. Conditions for gaseous loss of nitrogen through nitrification-denitrification existed at the surface of the model as evidenced by low oxidation-reduction potentials during periods of saturation and high potentials after drainage. Nitrogen and potassium were removed from surface water before traveling half the model length. Hence, nitrogen loading could be increased, and the resulting increased grass growth would probably improve treatment effectiveness for other elements. The loss of 25 percent of the applied phosphorus in runoff indicated a need for more surface contact, probably obtainable by increasing the model length. The subflow analyses showed excellent removal of nitrogen, phosphorus, and most heavy metals indicating effective removal mechanisms by the soil during the study period.

2.1.7.2 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, have 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 Carter, M.R., 1973. p. XIX-12], include the five general functions listed in Section 2.1.7 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 system 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 only influenced by extreme high and storm tides. While these species are usually found in close association, they can also be found separately.

The interaction of mangrove with the other parts of the ecosystem is complex and has been modeled by systems ecologists to varying degrees of detail and success. A simplified model of the nitrogen budget of mangroves was used by Steller [1976] in an analysis of various development alternatives at Marco Island. This analysis accounted for changes of mangrove biomass, detritus, and soil with time under the influence of sunlight and tide. Lugo, et al [1971, in Carter, M.R., 1973, pp. XIX-11 to XIX-16] considered:

1. oxygen, rain, river flow, tides, mixing action, sunlight and heat as forcing functions,
2. consumers, detritus, plankton, phosphorus and the influence of brackish water, to name just a few of the ecosystem components, and
3. the terrestrial and aquatic food chains and aerobic and anaerobic processes in the mud as outputs.

Stanford [1976] developed a model of nutrient cycling in a south Florida mangrove system which showed that the system acted as a source for organic carbon to, and a sink for nitrogen from, the contiguous estuary, while maintaining an approximate balance in terms of phosphorus.

Wharton, et al [1976] has presented suggestions for "ways of inserting housing in mangroves". He states that housing must not interfere with water circulation, and proposes that dwellings be constructed on pilings above the expected maximum stormtide elevation. He also mentions that there is a zone on the landward side of mangrove swamps in which, after exceptionally high tides or heavy rains, salt water pools will remain which provide breeding grounds for mosquitos.

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

2.1.7.3 <u>Turtle grass</u>

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 initial 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.

Marine grasses, which are usually completely covered at all stages of the tide, will stand roughly vertically at slack tide. As the tidal current increases, the grasses will begin to bend, thereby decreasing their overall frictional resistance to the flow. The Manning coefficient, n, for flow through grasses will initially increase (in the range $0.2 \leq n \leq 0.4$) for extremely low flows, but at some transitional depth (about 0.5 ft depth for Bermuda grass [Palmer, 1947]) Manning's n will decrease toward the more normal value of about 0.03 for flow in open channels. A Manning's n value of 0.3, the average value for Palmer's function as applied to Bermuda grass at low flows, may be converted to a Nikuradse's equivalent sand roughness, k, value from the relationship

$$M = \frac{46.8}{k^{1/6}} = \frac{1.486}{n}$$

thus, (2.4)

$$k = \left[\frac{46.8\ n}{1.486}\right]^6 \text{ ft}$$

$$= 711{,}000 \text{ ft}$$

for depths of flow less than 0.2 ft. For a depth of 2 ft, Manning's n as given by Palmer for Bermuda grass has decreased to a value of 0.032 (k = 1.05 ft). Thus at normal tidal depths in canals the additional effect of grasses on the flow is negligable. The extremely high value of k corresponding to n = 0.3 seems to indicate that the Manning Formula is not valid in this range, or at least that it should be used with the greatest caution.

The production, seasonal variation, and salinity tolerance of *Thalassia* 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^{o}C. Its salinity tolerance range is from 3.5-5 ppt up to as high as 60 ppt for short periods of time, and its temperature range is about 20 to 35^{o}C [Zieman, 1975, in Cronin, 1975, p. 553]. *Thalassia* stocks fluctuate with the season, but normally do not completely die out in winter.

2.1.7.4 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. Dredging has not come to an end, and while large scale construction of residential canal systems on marshlands is not likely to occur in the future, the maintenance of navigation is still essential, and will continue to require dredging in these sensitive areas. 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].

2.2 Field Observations and Analysis for Development of the Model

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 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.

2.2.1 Preliminary Site Investigations and Field Surveys

The preliminary site investigations and field surveys conducted by the Hydraulic Laboratory from 1974 through 1977 are summarized in Table 2.3. The hydrographic surveys 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 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.

The Cudjoe Gardens and Venus Waterway hydrographic surveys in 1974 and 1975 were undertaken to obtain experience not only in analyzing flow characteristics in existing systems, but also in evaluating design alternatives available for improving these systems. In 1975, the Frenchman's canal and 57-Acres project sites provided locations for measurements of longitudinal dispersion in a simple system (the former) and in a complex canal network (the latter), as well as the preliminary investigative work which was to lead later to the detailed flow studies. The 1977 field studies at the 57-Acres and Loxahatchee River sites provided detailed measurements of the effect of wind and salinity gradients on the flow in these particular canals, which were used to calibrate and verify the three-dimensional model.

2.2.2 Instrumentation and Support Equipment

The major instrumentation used by the Hydraulic Laboratory for canal surveys will be listed here and described in more detail in Chapter 12. Manufacturer's names and equipment model numbers are mentioned so that the reliability of the data may be assessed by other investigators familiar with this kind of equipment.

1. *Tide Recording*

   A Leupold-Stevens type F, model 68 water level recorder mounted on a stilling well was placed near the shoreline. The stilling well was either pushed into the bottom and braced as a free-standing structure, or was mounted on a bulkhead or a dock. All tide records were leveled to the nearest benchmark which could be referenced to the National Geodetic Vertical Datum and thereby to MLW.

2. *Depth Recording*

All bathymetry was recorded using a Benmar echo sounding recorder Model DR-68, 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 ft increments.

3. *Distance*

All distances have been taken with a metal or fiberglass 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.

4. *Current Speed and Velocity*

In the early variability studies and design modification surveys, an Ott Model C.1 laboratory propeller meter or a Snyder Oceanography Services, Inc., Savonius-rotor meter was used to measure current speed. All velocity measurements after April 1975 used a Cushing dual-axis electromagnetic current meter, model 82-CP velmeter sensor with model 632-P portable converter. These were supplemented by four more of the same kind of meter in February 1976. The velocity probes were each mounted on a separate tower which was set up on the centerline of the channel. The probe was then moved up and down the tower by means of the carriage to which it was attached, so that vertical profiles of the horizontal velocity vector could be obtained. Occasionally the probe would be turned ninety degrees so that the vertical, instead of the transverse, flow component could be measured along with the longitudinal component.

5. *Wind Speed and Velocity*

In October 1976 a hand-held low-speed anemometer, Davis model A/2-4", was purchased for measuring wind speed. This was used to obtain samples of wind speed at about ten ft off the surface of the water, and indicated that a substantial wind effect might be present. Accordingly, an R.M. Young windvane and three-cup anemometer, with a sampling recorder, were purchased in October 1977 and used on one field trip to the 57-Acres site. The wind velocity was measured at an elevation of ten m near the north bank of the south loop, near station A (Figure 2.8).

6. *Salinity*

Salinity has been measured by titration and also has been calculated from conductivity and temperature observations. For titration a LaMotte test kit Model POL-H code 7459 was used, while conductivity was obtained by use of a LaMotte conductivity meter Model DA. Considerable differences between readings by the two methods convinced the investigators to rely on the titrations.

For data reduction a linear regression was fitted to ten readings taken simultaneously by both methods and the resulting correction was applied to all conductivity measurements.

7. *Water Temperature*

Water temperature was measured with both a mercury thermometer, and a LaMotte temperature meter with a thermistor probe, model KA. During the June 1977 Loxahatchee north canal survey the readings compared with each other within $2^{o}C$ and since neither instrument showed more than $2.4^{o}C$ temperature difference between surface and bottom (corresponding to a density difference of about 8×10^{-4} ppt) it was decided that either instrument would give satisfactory results as long as the readings were only taken with one instrument at any one station.

8. *Dye Concentration*

The concentration of Rhodamine WT water tracing dye was measured with a Turner Designs Model 10-005 field fluorometer, either in the flow-through mode or the sampling mode depending on the data required. If a profile at one depth throughout the canal system is desired, the fluorometer is arranged to continuously measure the concentration of the water as it is pumped up through the machine from the required depth. If, on the other hand, many discrete samples at various stations and depths are required, and salinity is being measured at the same time, it is better to set the fluorometer up for measuring the concentration in a cuvette.

2.2.3 Dye 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 unidirectional flow (e.g. in rivers [Fischer, 1967]) and that some work had been done to measure diffusion and dispersion 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.

2.2.3.1 Longitudinal Dispersion

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 which 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 convective-dispersion equation for turbulent flow (See Chapter 3) it is assumed that the dispersion process can be described approximately 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

$$c(X,t) = \frac{M}{\rho A \sqrt{4\pi E t}} \exp\left[- \frac{(X - \overline{U}t)^2}{4Et}\right] \tag{2.5}$$

where M = mass of pollutant injected, [M]

c = concentration, dimensionless

ρ = density of solution, (M/L^3)

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

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

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

t = elapsed time since injection, (T)

$\overline{U}$ = mean (steady, uniform,) velocity of flow 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 $X = \overline{U}t$ and decreases with time according to

$$c_{max} = \frac{M}{\rho A \sqrt{4\pi E t}} \tag{2.6}$$

The centroid of the dispersing cloud is also at $X = \overline{U}t$, and the variance of the distribution of concentration is given by $\sigma^2 = 2\ Et$. [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 from measurements of tracer distributions. One form of the dispersion coefficient E is [Taylor, 1954]

$$E_\ell = KR\ |u^*| \tag{2.7}$$

where

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

K = dimensionless dispersion coefficient

R = hydraulic radius (L)

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

The dimensionless dispersion coefficient K is an empirical coefficient which is independent of the depth and roughness of the channel, but a function of the regularity of the channel; low values of K correspond to straight regular canals while high K values 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, if Equation (2.5) is to be used to evaluate the dispersion coefficient, there is a further limitation described by Fisher [Nov, 1967, pp. 192, 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 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 2.7) is that the distance downstream from the point of tracer injection to the sampling point should be [Fisher, Nov, 1967, p. 213]

$$L > 1.8 \frac{\ell^2}{R} \frac{\overline{U}}{u^*} \tag{2.8}$$

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)

R = hydraulic radius, (L)

$\overline{U}$ = mean velocity, (L/T)

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

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, special considerations will be required to overcome this restriction.

2.2.3.2 Longitudinal Dispersion Coefficients

There are a number of different ways of analyzing tracer concentration profiles to obtain a longitudinal dispersion coefficient [Fischer, Nov, 1967]. Each requires a particular set of measurements, so it is necessary to decide before going to the field exactly what measurements the canal designer needs. The method used by the Hydraulic Laboratory in 1975 is called the "modified semi-log plot" method, originally described by Holley and Harleman [1965, p. 110], based on Equation 2.5. 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.

2.2.4 Implications of Results of Field Experiments

2.2.4.1 Salinity and Temperature Gradients

In the early variability studies, during September 1975, salinity and temperature profiles were measured in one of each of the four pairs of canals at Port Charlotte, Punta Gorda, Pompano Beach, and Loxahatchee River [Hydraulic Laboratory, Feb. 1976]. Such profiles can indicate the presence or absence of density gradients which could lead to stratification, and in some cases it is possible to clearly discern the shape of a density wedge from a plot of density profiles at several stations. In general, one expects to find some degree of stratification 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.

A salinity profile is adequate for showing the shape of a density profile provided the water temperature is relatively constant. However, as shown in Figure 2.22, the density of sea-water varies with both salinity and temperature, and therefore density should be used when 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.

In June, 1977, a field survey was conducted at the Loxahatchee north canal for the express purpose of investigating the relationship between the density structure and the layered flow that was known to exist there. In addition, a dye dispersion study was conducted to see whether vertical dye concentration profiles could be used in any way to obtain dispersion coefficients. The results of this study in terms of plots of density, velocity, and dye concentration are given in Appendix A.

Some of the vertical density profiles definitely show that a density wedge is present (for example, Figure 2.23) while those with relatively uniform gradients indicate locally poor mixing but no stratification at that time (Figure 2.24). Figure 2.25 shows relatively complete vertical mixing in the half near the mouth.

2.2.4.2 Wind and Velocity Profiles

A field survey was conducted at the 57-Acres canal network in October, 1977, to measure the local wind and corresponding vertical water velocity profiles over an extended period. The wind was recorded from October 18 at 1500 to October 21 at noon. Velocity profiles were taken at the five stations shown in Figure 2.26 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 2.27. The wind data and water velocity profiles are on file at the Hydraulic Laboratory. An analysis of the effect of wind on circulation in the canals is included in Chapter 4.

2.2.4.3 Longitudinal Dispersion Experiments

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 method of injecting the tracer, establishing the sampling station, and analyzing the plotted tracer concentration is described in Morris and Christensen [1976a, pp. 74-89]. The results are summarized in Table 2.4.

In general it was found that the dimensionless dispersion coefficients K were very low, in the range of 2 through 20, in comparision to values of from 50 to 700 found by Fischer [1967, p. 188] in natural rivers. One possible explanation for low values of K is the low velocities which are found in Floridian canals, typically less than 0.8 fps and commonly on the order of 0.1 fps, since the longitudinal dispersion coefficient is defined as a function of the velocity and concentration spatial fluctuations.

Two field surveys to obtain data on dispersion in a more complex network over several tidal cycles were conducted in July and October, 1977, in the 57 Acres canal network. These spanned the periods of July 18 through 22 and October 16 through 22. The types of measurements taken on each survey have been summarized in Table 2.3 (Section 2.2.1) and in Appendix B. Both two- and three-dimensional plots of the centerline dye concentration 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, since before every run the blank and span on the fluorometer were adjusted using the same solutions, which were stored in opaque containers. At all locations which were sampled a value above zero concentration was detected, and when the operators decided that they had been measuring a constant value for some time, such as the length of a reach, the sampling procedure was terminated. Thus, any reach on the map with zero indicated concentration is a location where no measurements were taken.

Figure 2.28 is a map showing the locations of the stations used to designate reaches for the dispersion studies. The two-dimensional plots are subdivided by reaches designated by the endpoint stations. Thus, reach AD is the south loop to its intersection with the south straight.

The conditions for the first dispersion study are summarized in Appendix B. This survey was intended as a feasibility study to determine whether a centerline dye concentration measurement from one depth could be carried out over several tidal cycles, and could provide useful data. The nine three-dimensional plots, two representative examples of which are given in Figures 2.29 and 2.30, 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 disappears 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,

possibly 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 (see Appendix B for a summary of these values). 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.

In spite of the rather incomplete results of the first dispersion survey, a second similar dispersion field survey was organized for October, 1977. This study, however, 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 determined to be negligable at that time. After several days of taking velocity profiles at five locations in the South Loop, two of the velocity meters were moved and the dispersion study began. Fifteen hundred ml of Rhodamine WT dye was released as a plane source across the channel in the upper 3 ft at high tide, and 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.

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 2.31) and the low tide at 1045 (Figure 2.32) appear to follow the expected trend of a reduction in the mass of dye throughout the system. Furthermore, portions of some of the other pairs of high and low, or low and high, plots *not near the dead-end* demonstrate reasonable behavior. 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 2.33) and during the following flood after the low at 771020/0950 (Figure 2.34) demonstrates moderate flushing. However, in the same two dye concentration plots (Figures 2.33 and 2.34) 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 2.35) 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 close enough to 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 mid-point 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 will be developed more fully in Chapter 12.

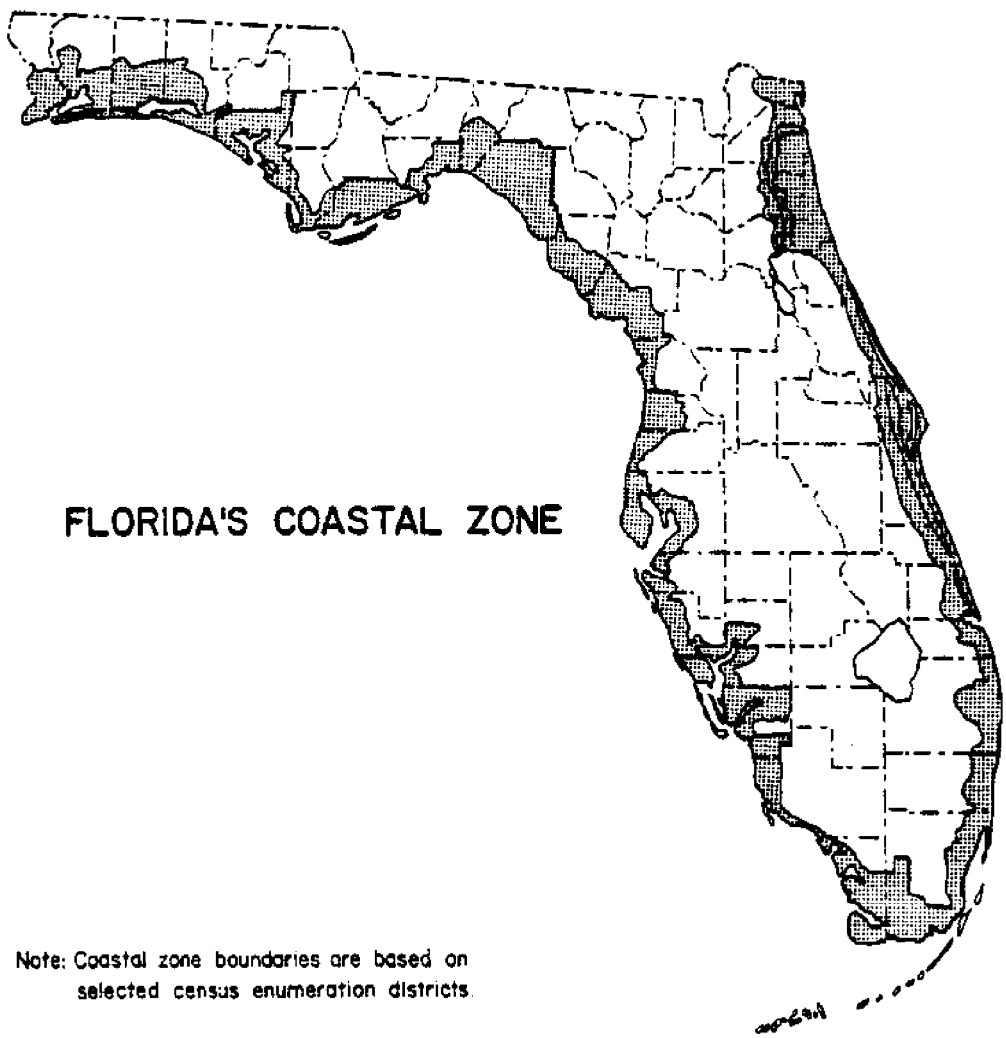


Figure 2.1 - Florida's Coastal Zone as Defined by the Florida Coastal Coordinating Council in 1971.

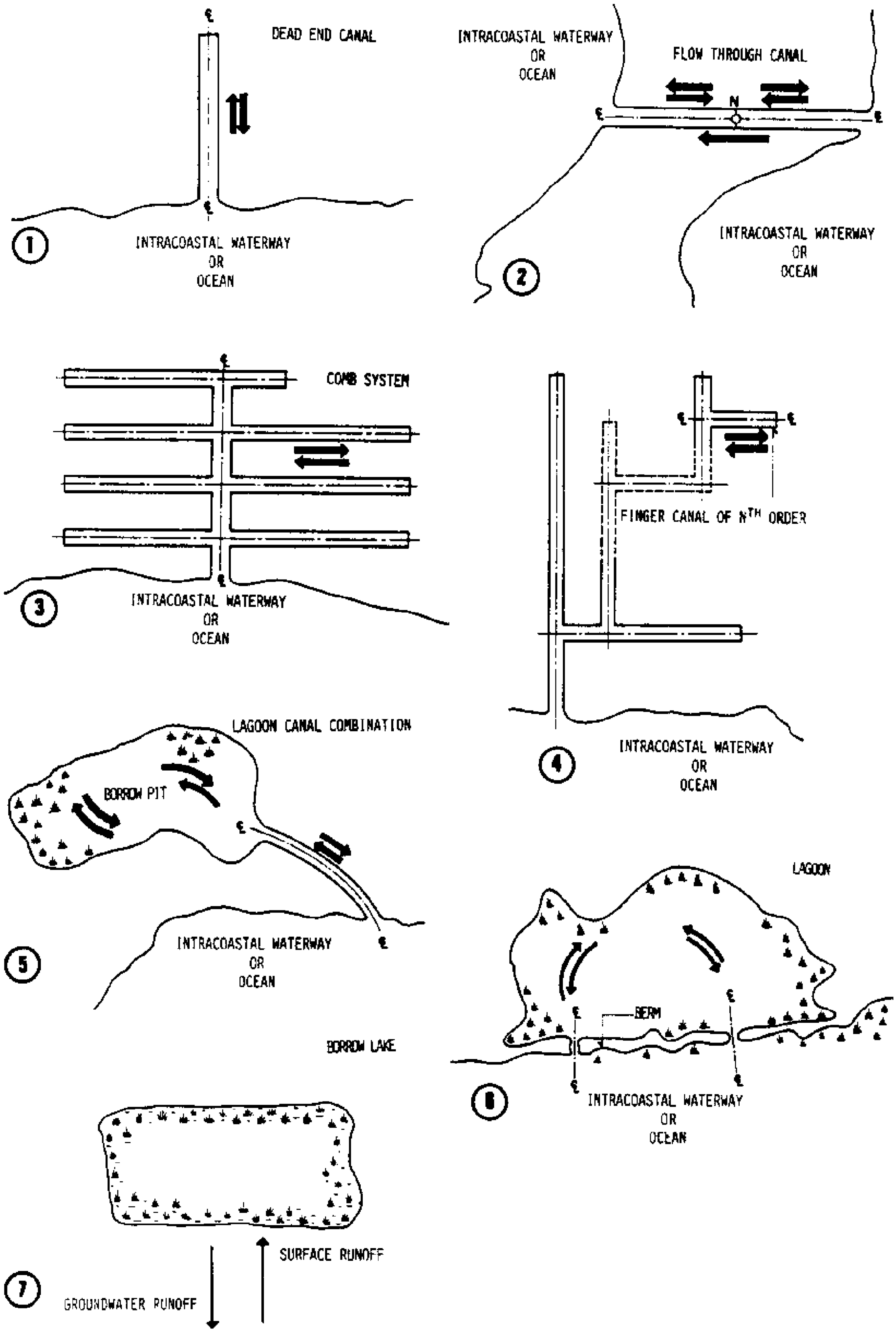


Figure 2.2 - A Geometric Classification for Types of Canals.

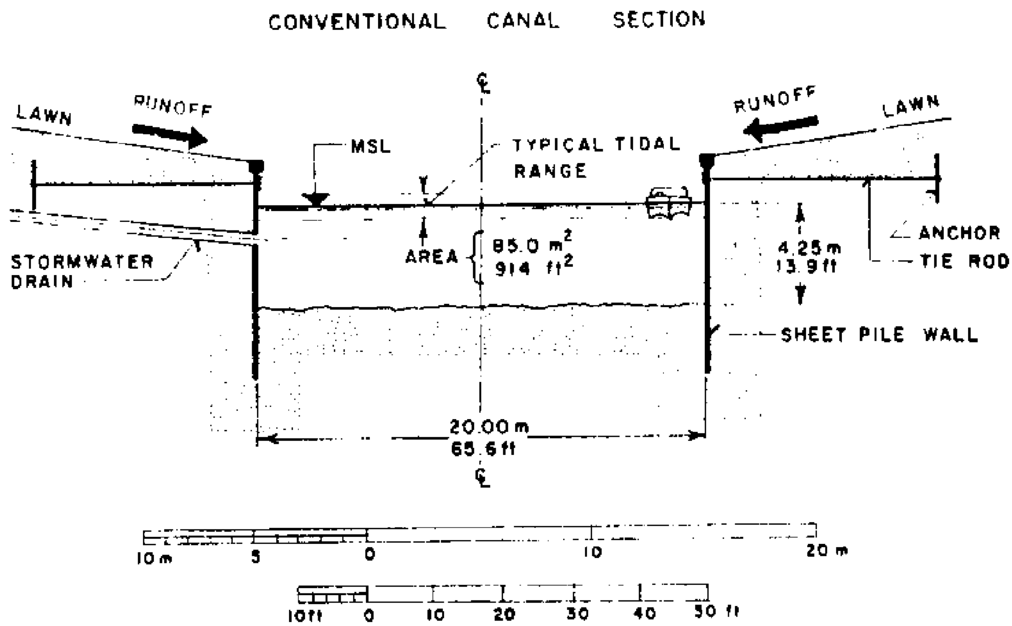


Figure 2.3 - Typical Cross-Section of Conventional Residential Canal (Source: Christensen and Snyder, 1978, p. 3).

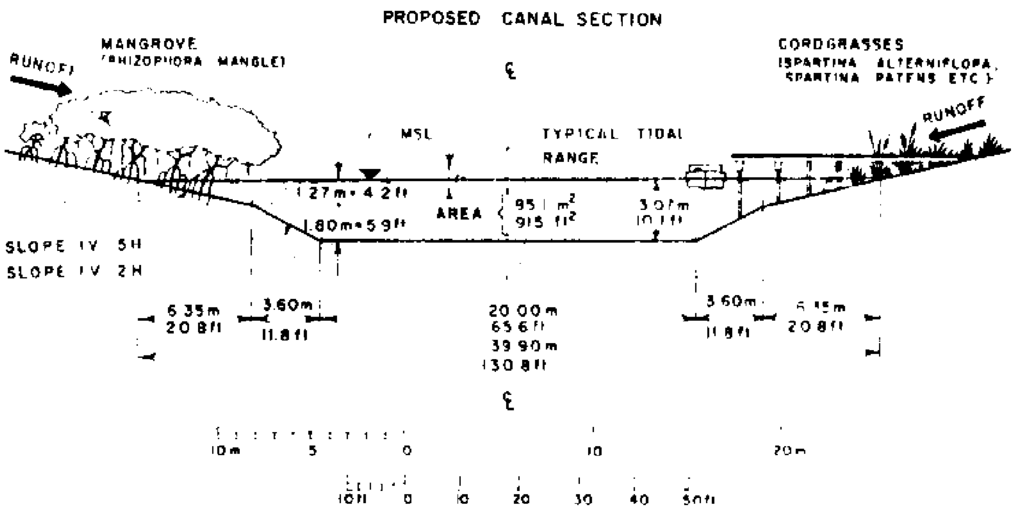


Figure 2.4 - Proposed Canal-Section With Same Area as the Conventional Canal Section Shown in Figure 2.3 (Source: Christensen and Snyder, 1978, p. 5).

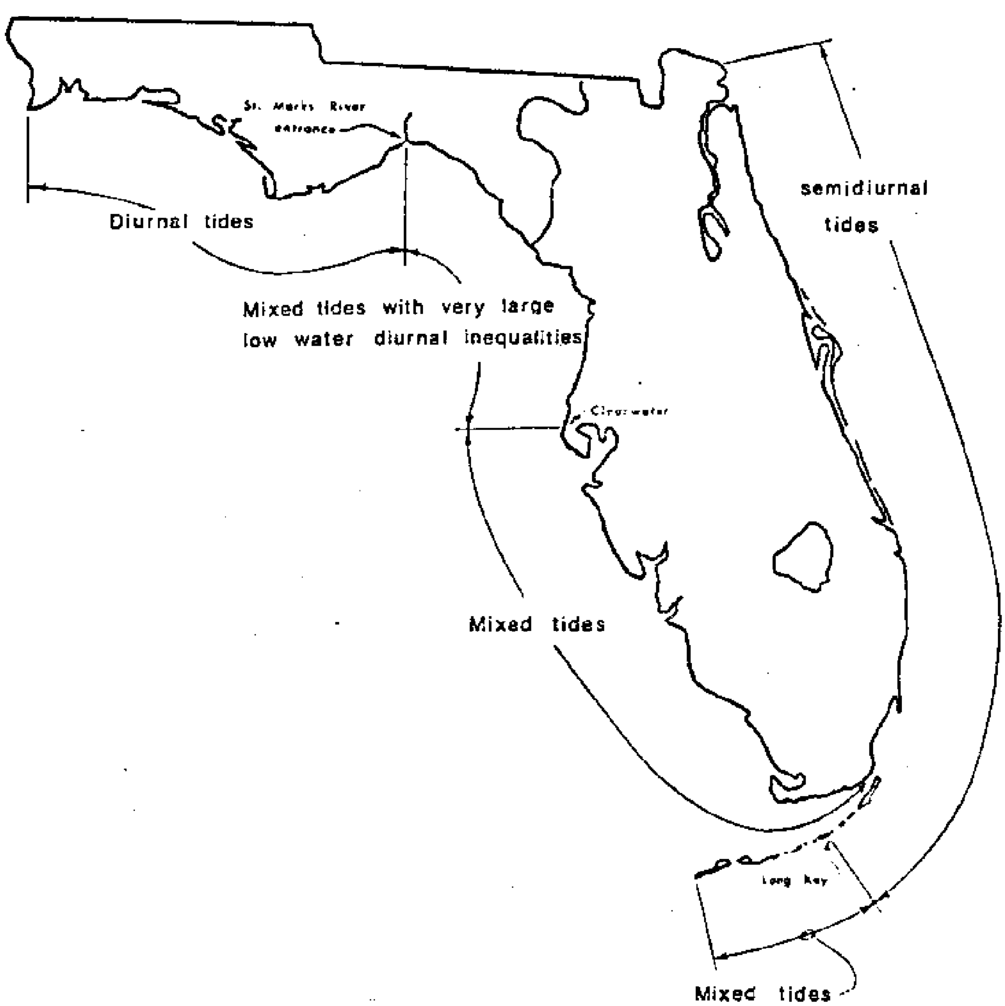


Figure 2.5 - Classification of Florida Tides (Source: Piccolo, 1976, p. 8).

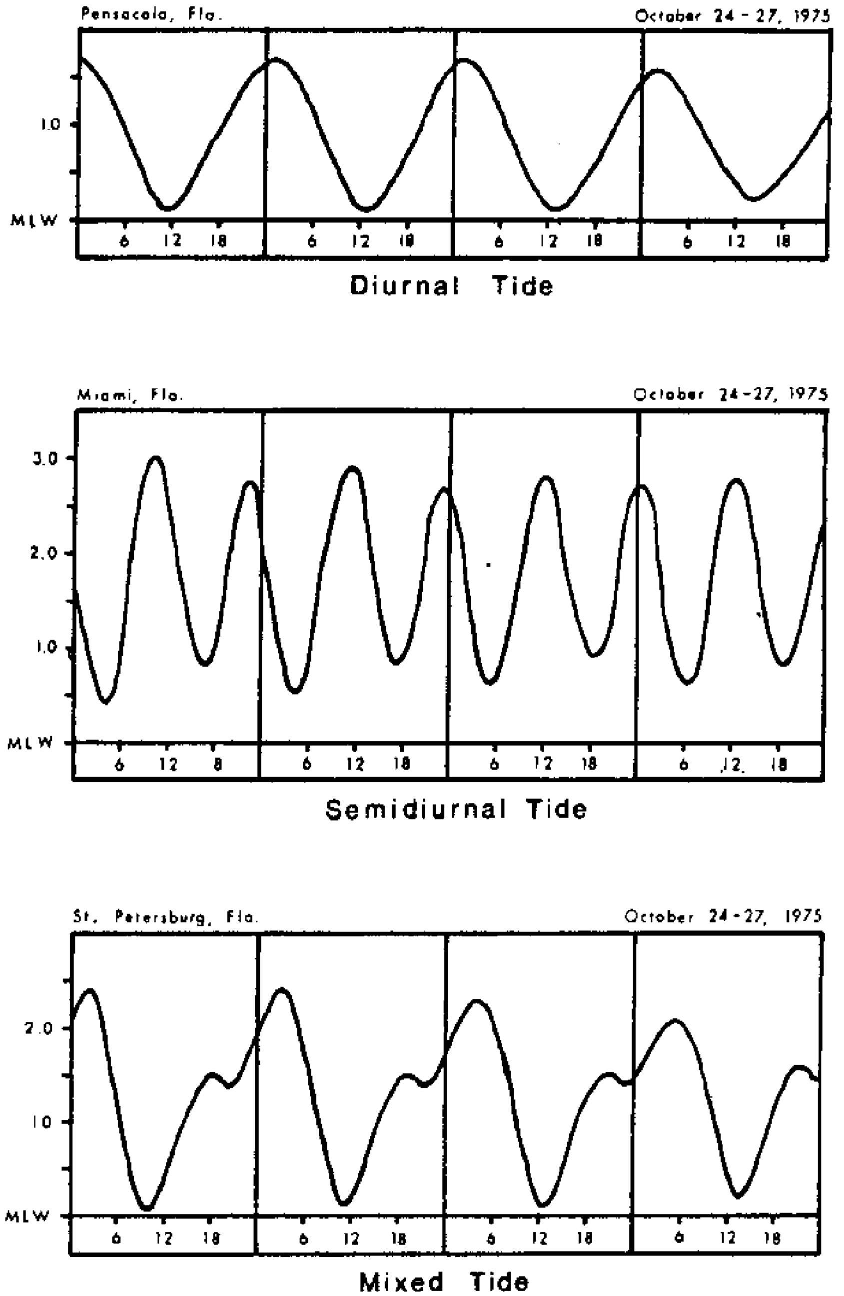


Figure 2.6 - Types of Tides (Source: Piccolo, 1976, p. 4).

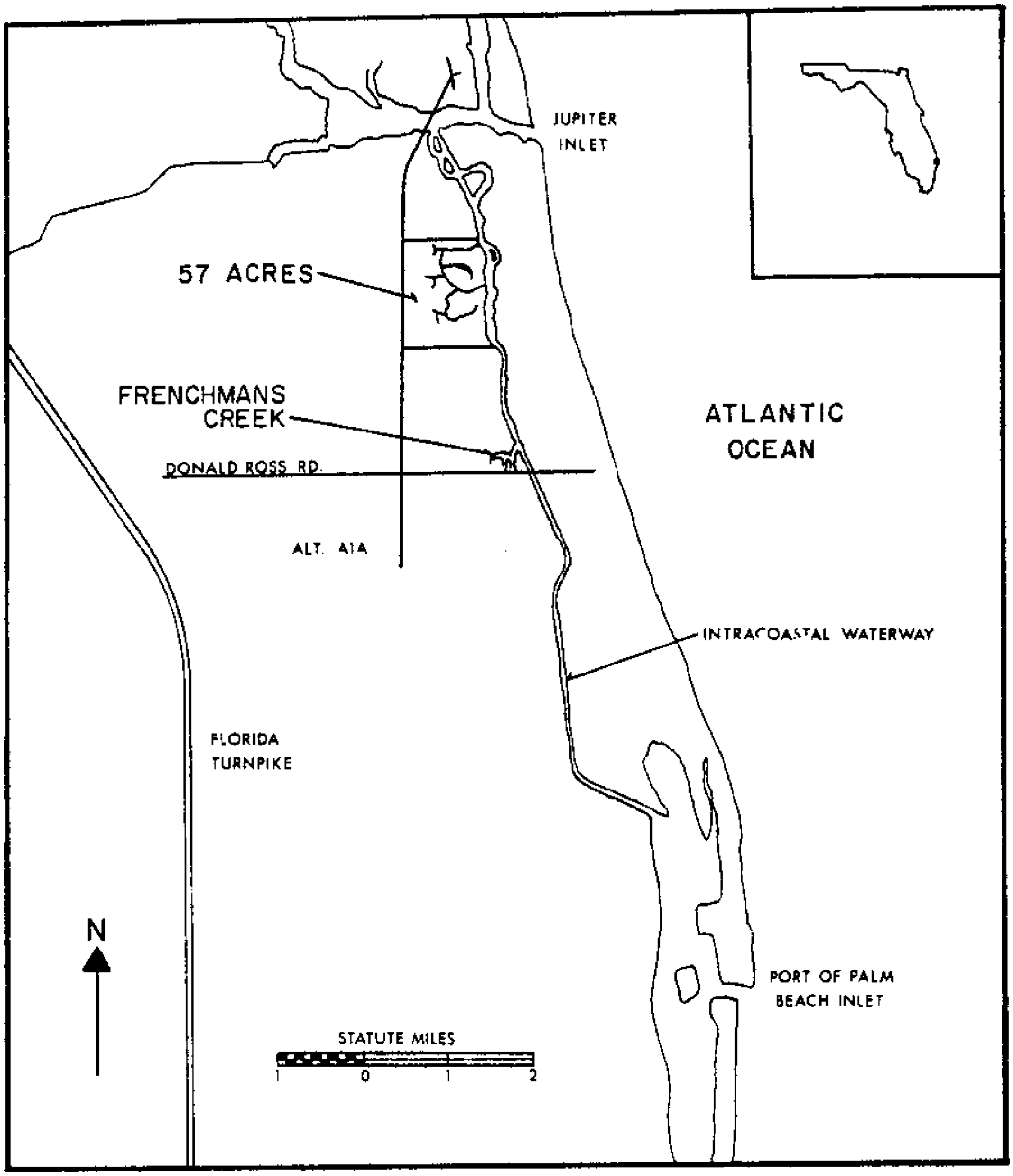


Figure 2.7 - Location Map for Frenchman's Canal and 57 Acres Canal Sites.

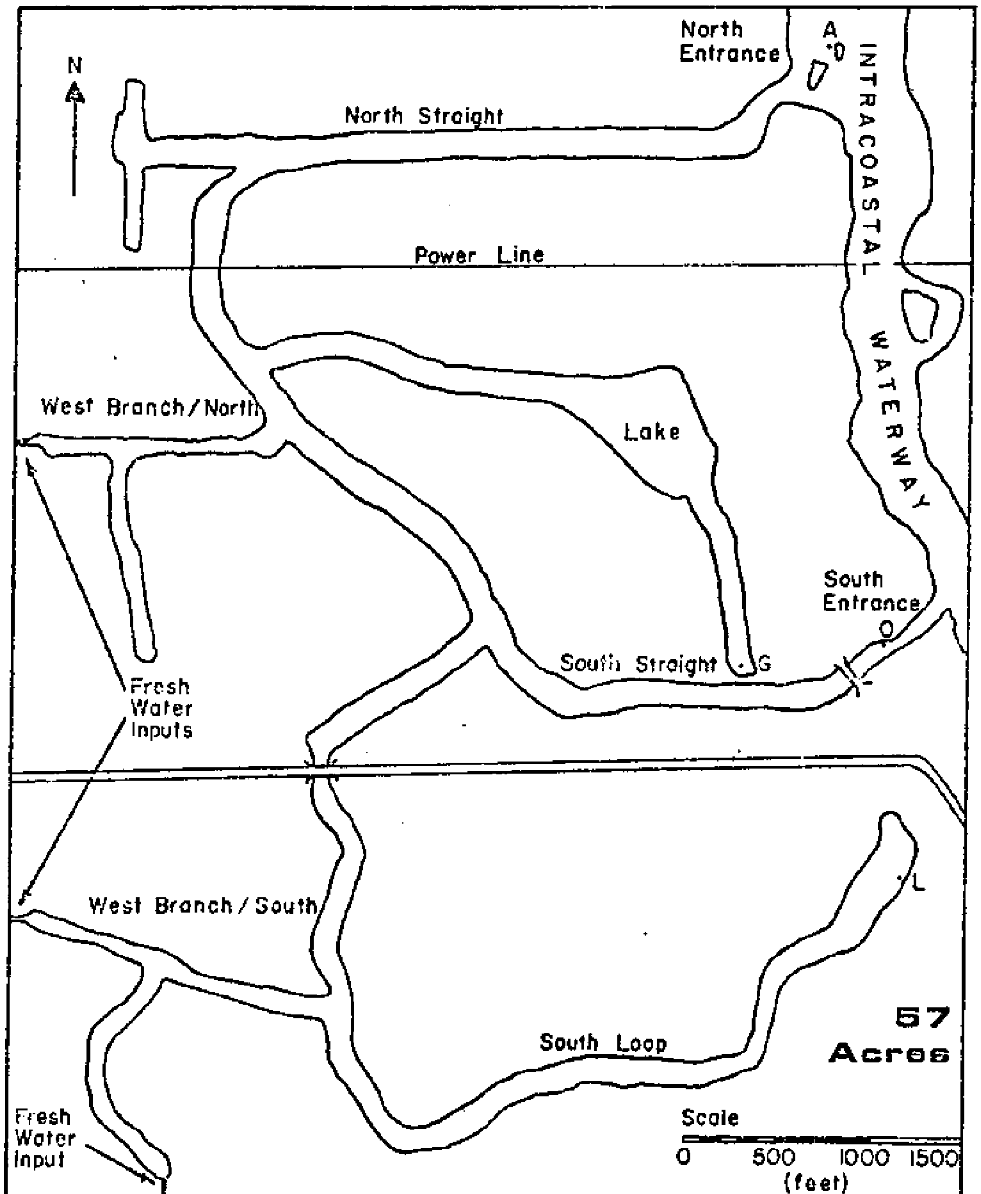


Figure 2.8 - 57 Acres Site Plan Showing Locations of Tide Gauges for September 1975 Measurements.

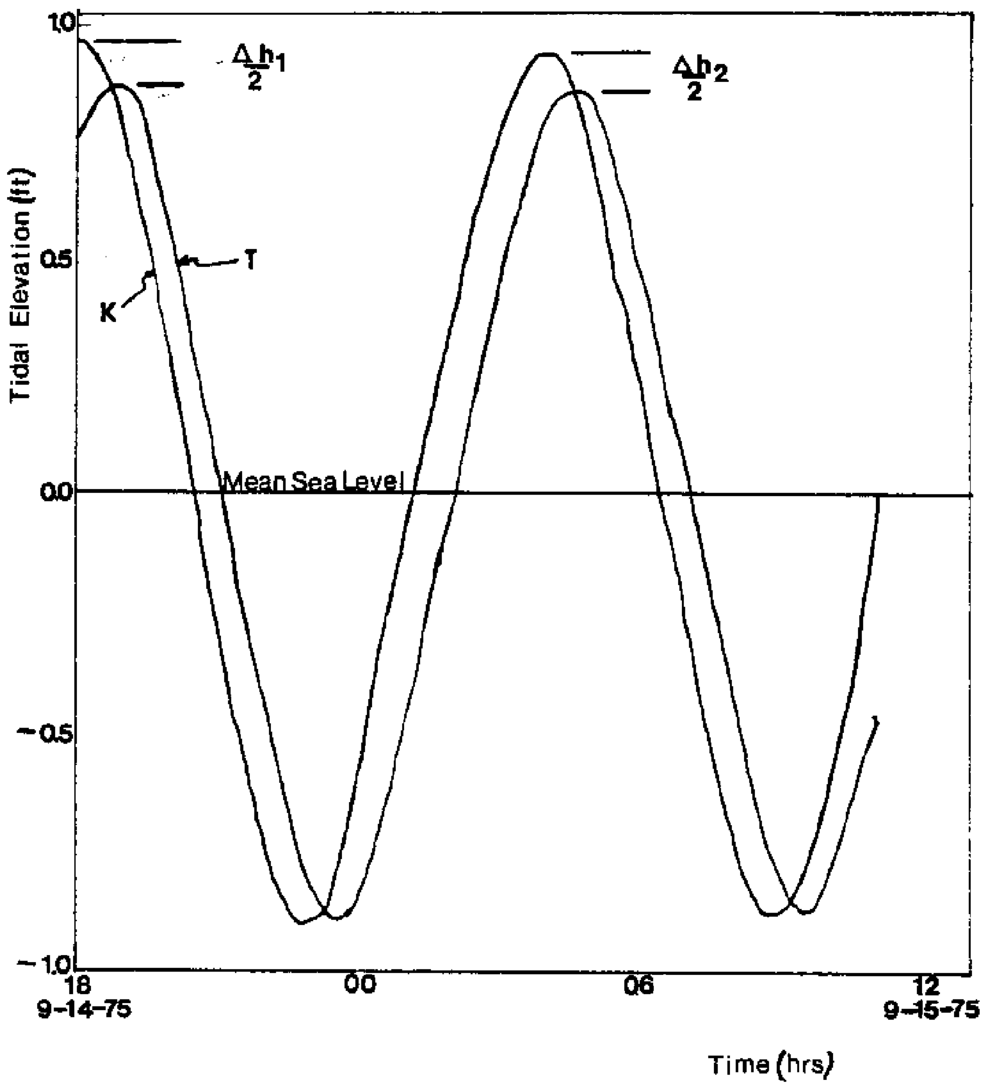


Figure 2.9 - Comparison of Tides at Stations K and T, 57 Acres Site, September 14 & 15, 1975. (Source: Walton, et al, 1975, p. 49).

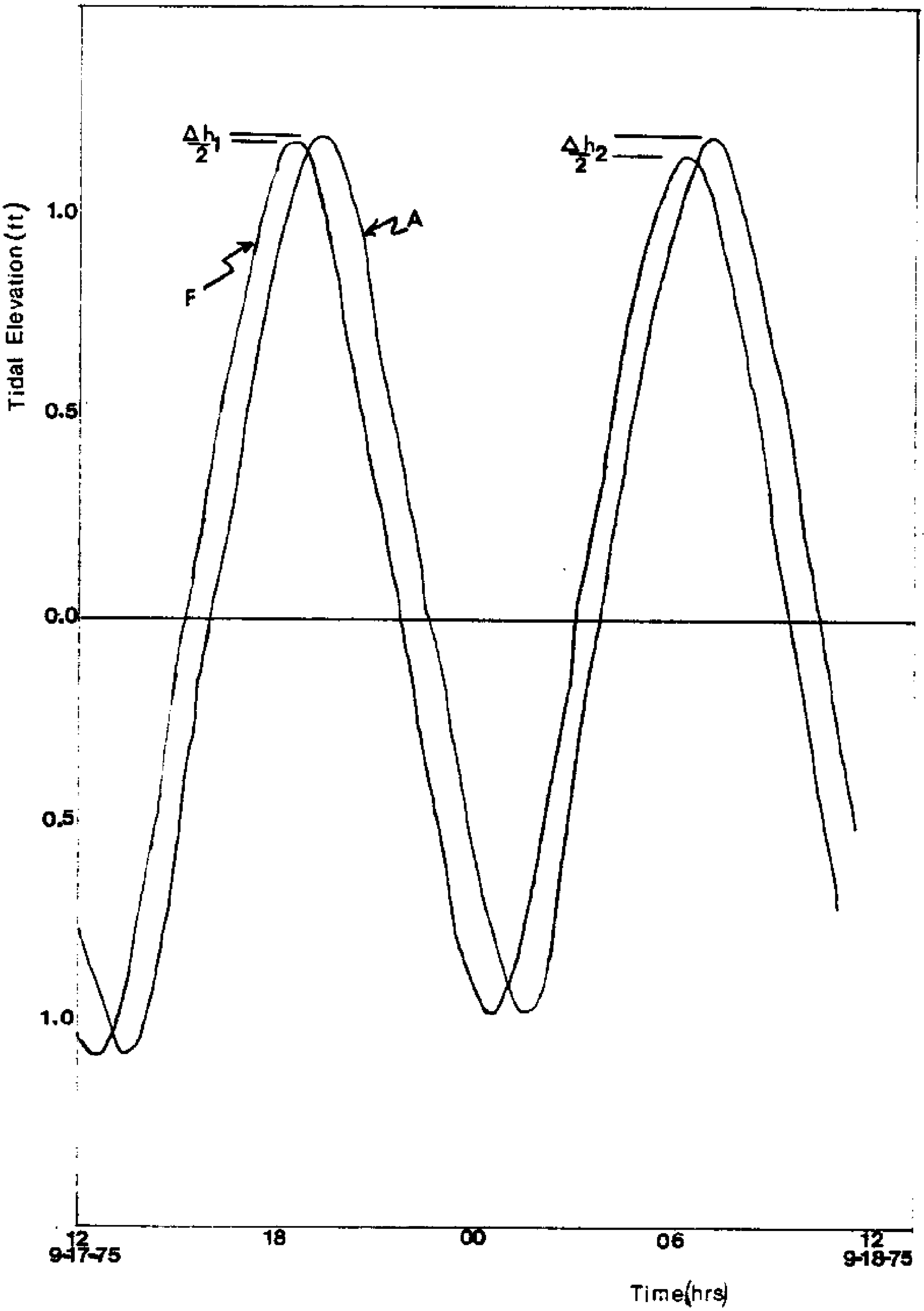


Figure 2.10 - Comparison of Tides at Stations F and A, 57 Acres Site, September 17 & 18, 1975. (Source: Walton, et al, 1975, p. 50).

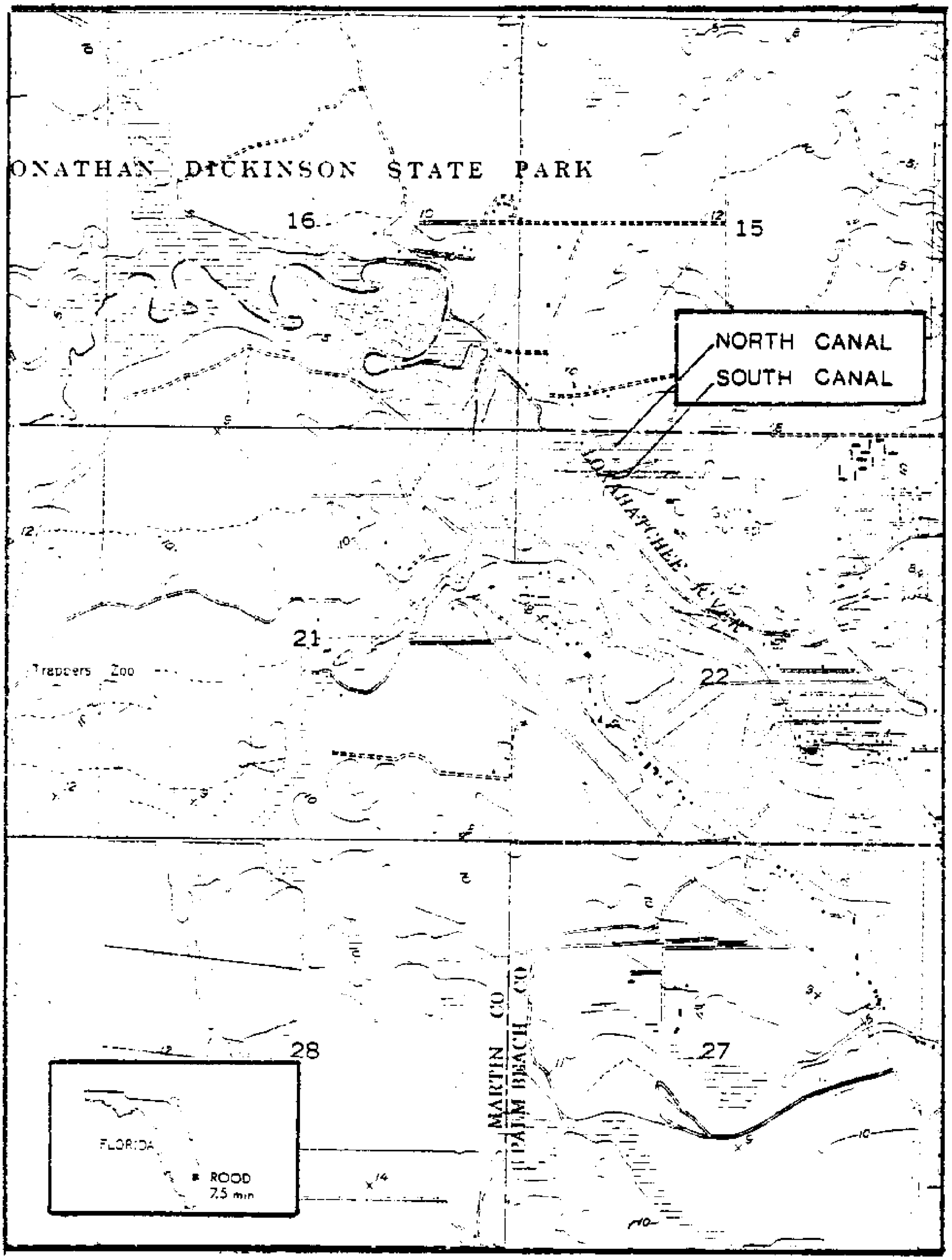


Figure 2.11 - Topographic (7 1/2' quadrangle) Map for Loxahatchee River Canals.

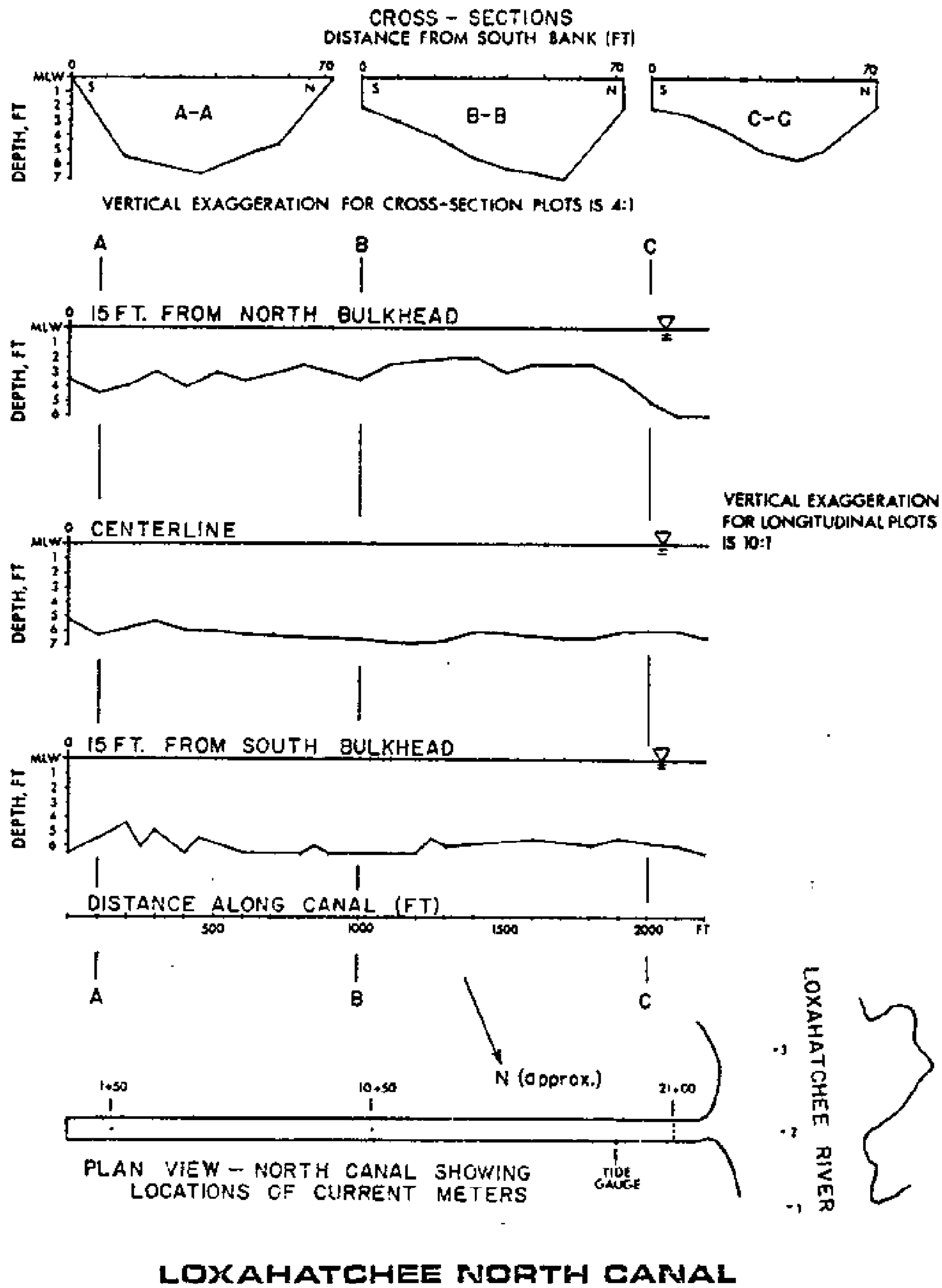
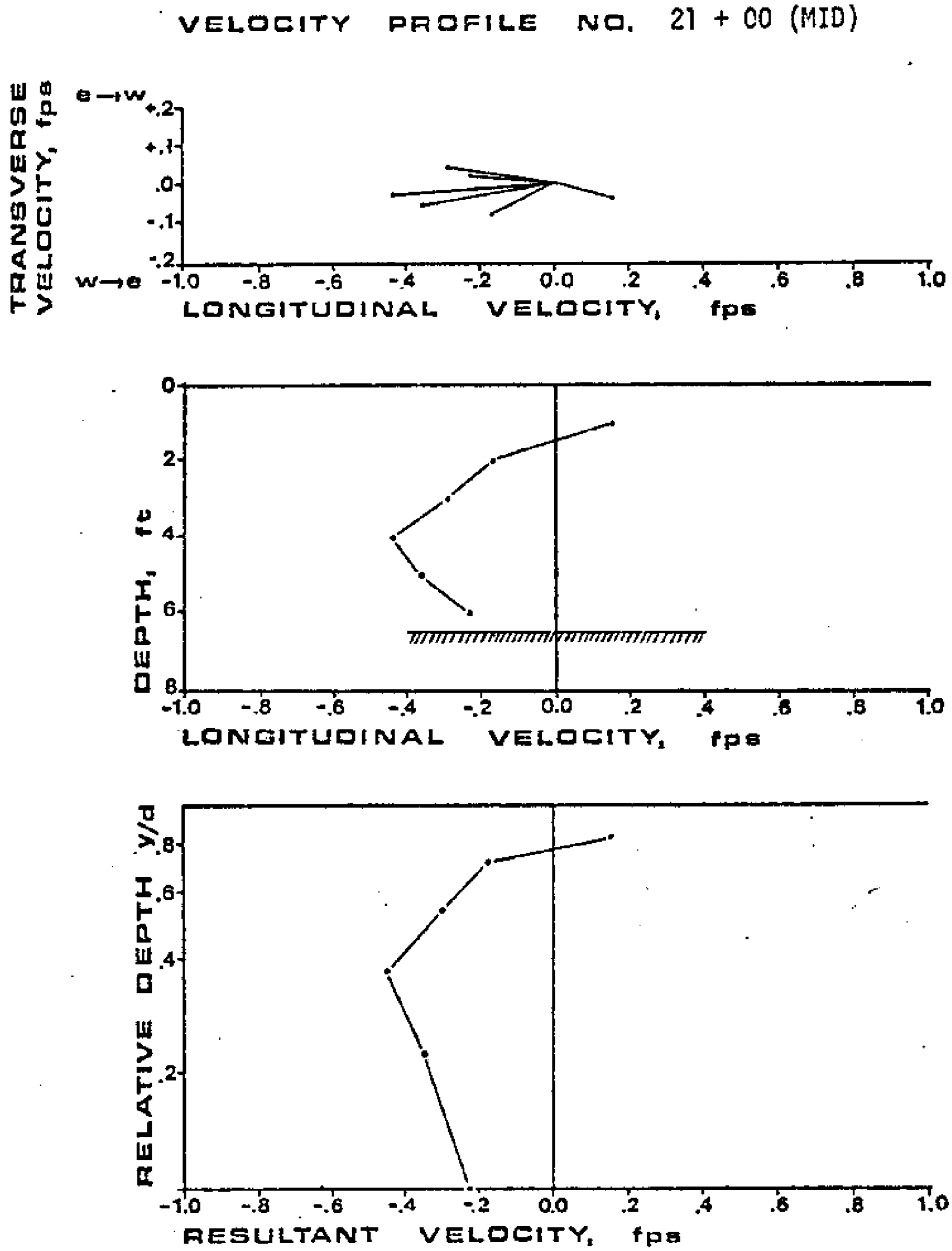


Figure 2.12 - Cross-Sections and Plan View of Loxahatchee North Canal, June 1977.



LOCATION: LOXAHATCHEE N. CANAL

DATE: 770613 TIME: 1747 TIDE: FLOOD WIND: 2 mph

Figure 2.13 - Velocity Profile Measured During Loxahatchee North Canal, Field Survey, June, 1977.

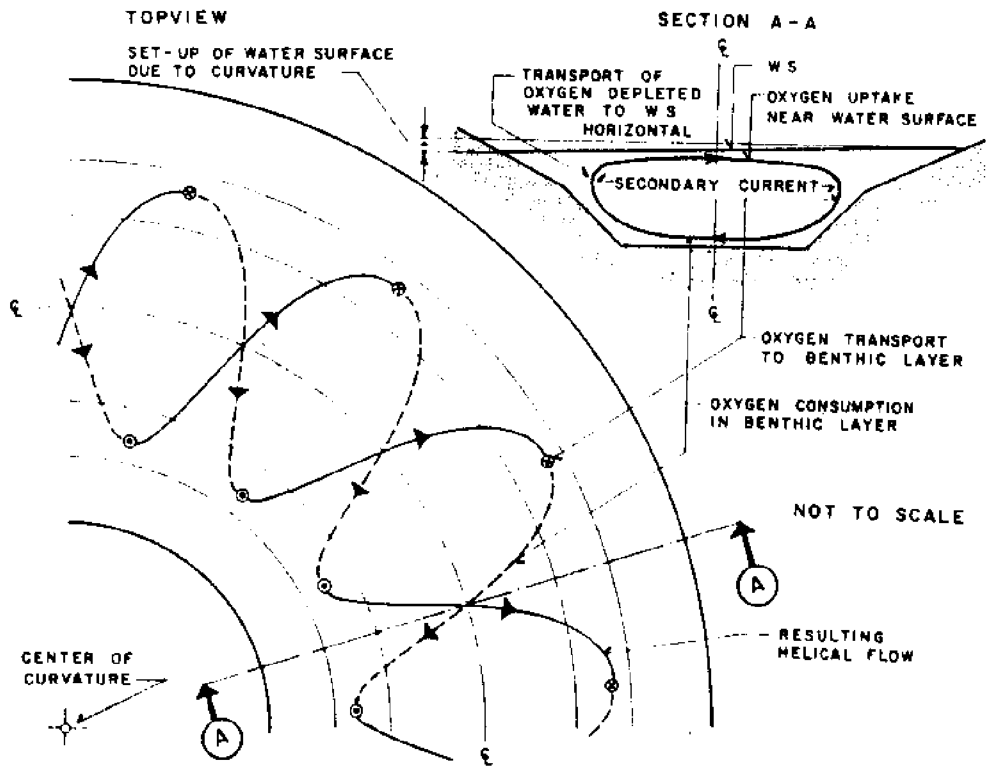


Figure 2.14 - Secondary Current and Resulting Helical Flow in Canal Bend. (Source: Christensen and Snyder, 1978, p. 6).

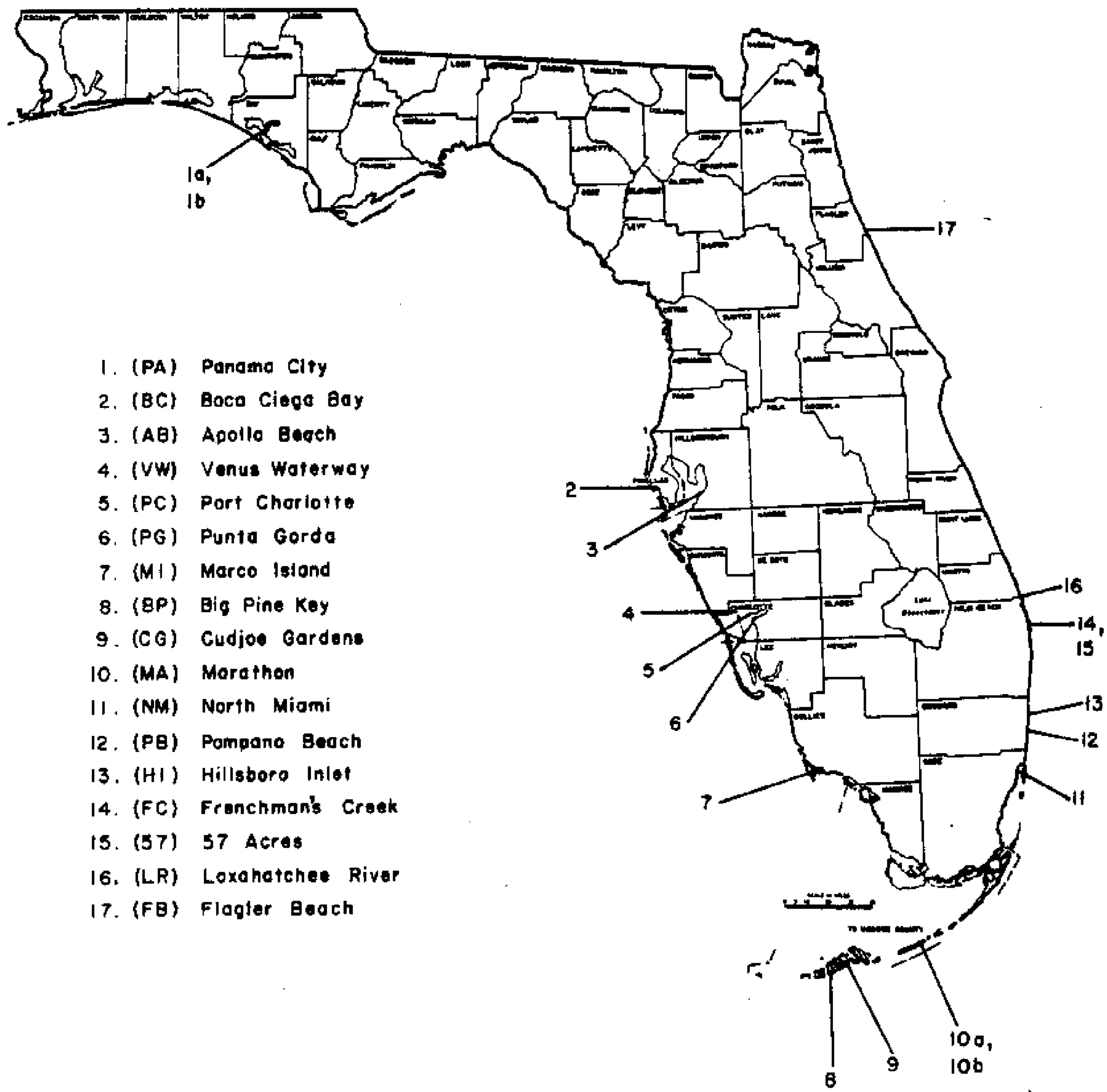


Figure 2.15 - Location Map for Sources of Data Used by Bailey [1976].

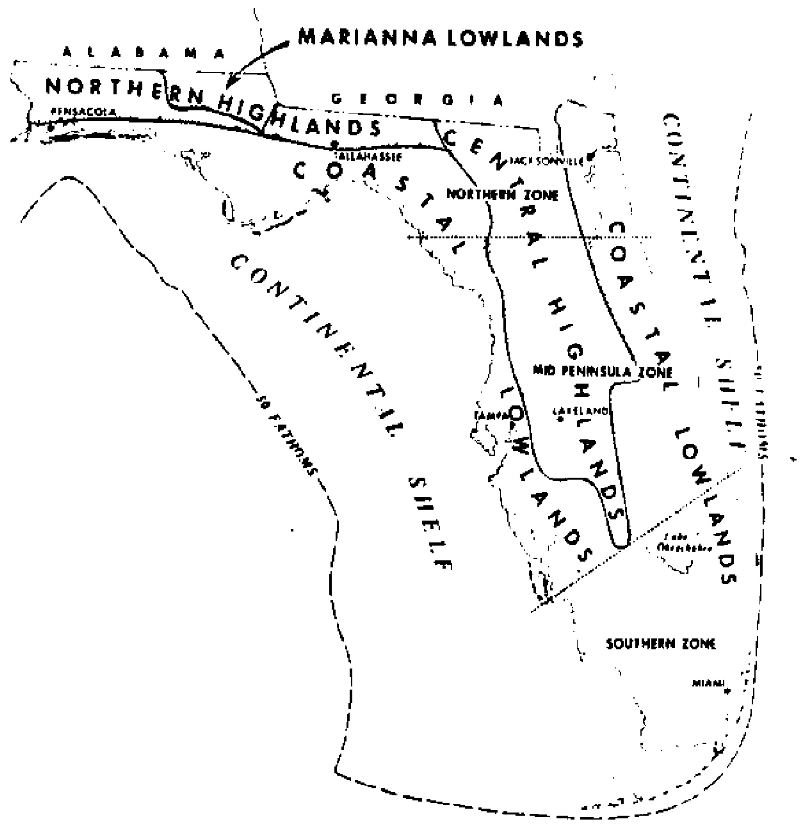


Figure 2.16 - Generalized Locations of Landforms in Florida (Source: Puri and Vernon, 1964, p. 8).

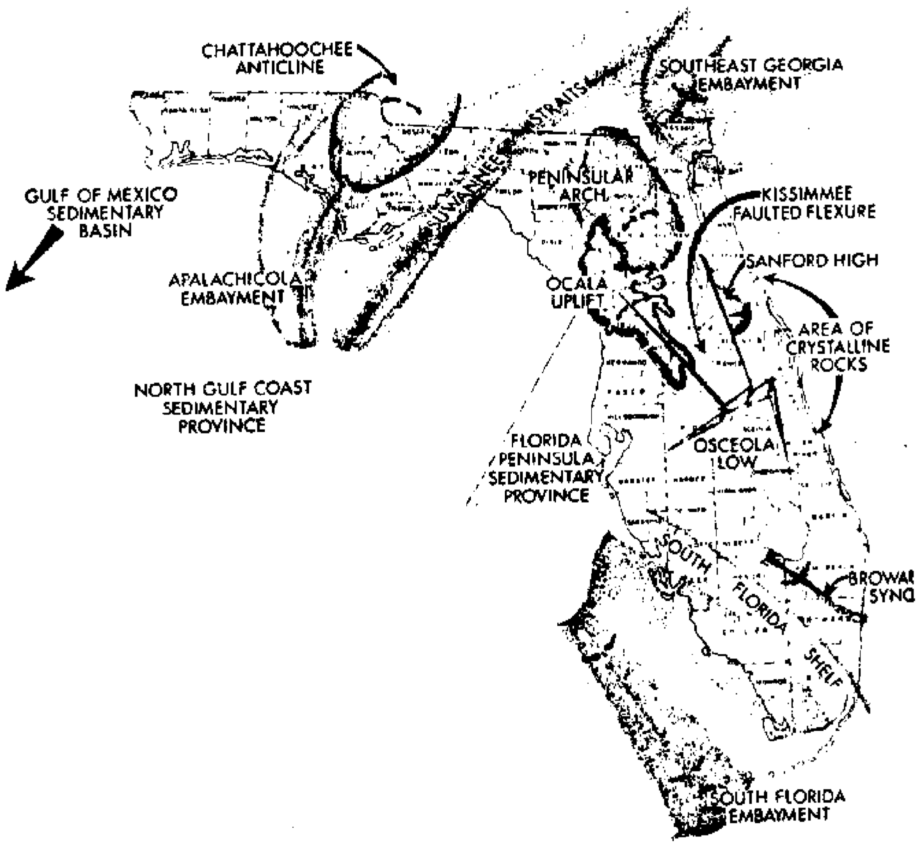


Figure 2.17 - Index to Principal Geologic Structures in Florida (Source: Puri and Vernon, 1964, p. 4).

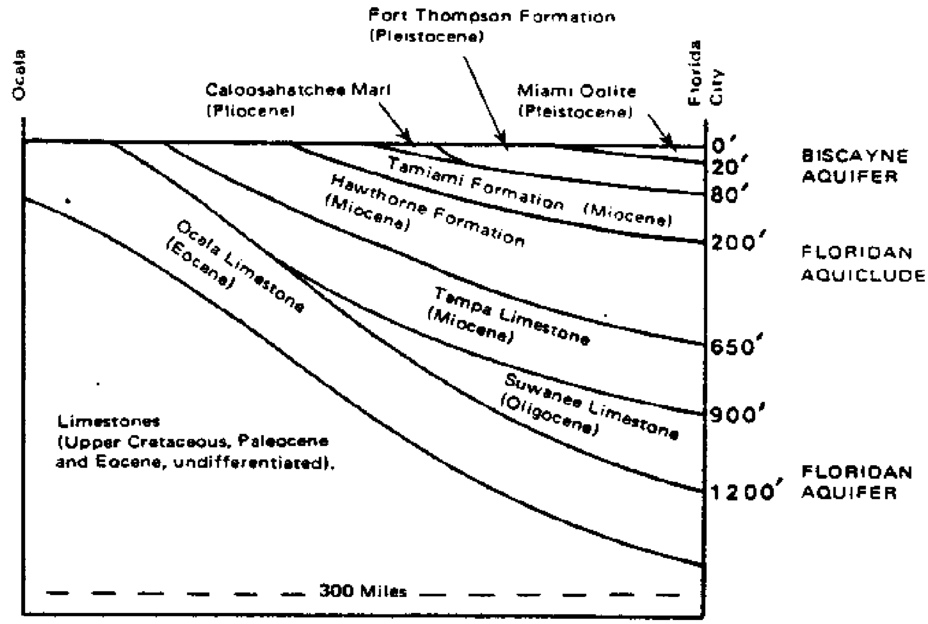


Figure 2.18 - General Layering of the Bedrock Formations Below Southern Florida from Ocala (North) to Florida City (South) (Source: Parker, et al, 1955, p. 63 in Veri, et al, 1975, p. 25).

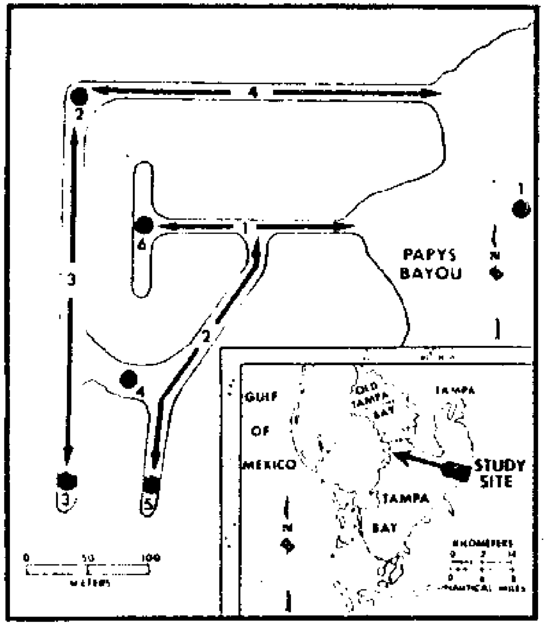


Figure 2.19a - Tampa Bay, Florida, Showing Location of Study Area. Hydrologic Sampling Stations are Shown by Arrows (Source: Lindall, Fable, and Collins, 1975, p. 82).

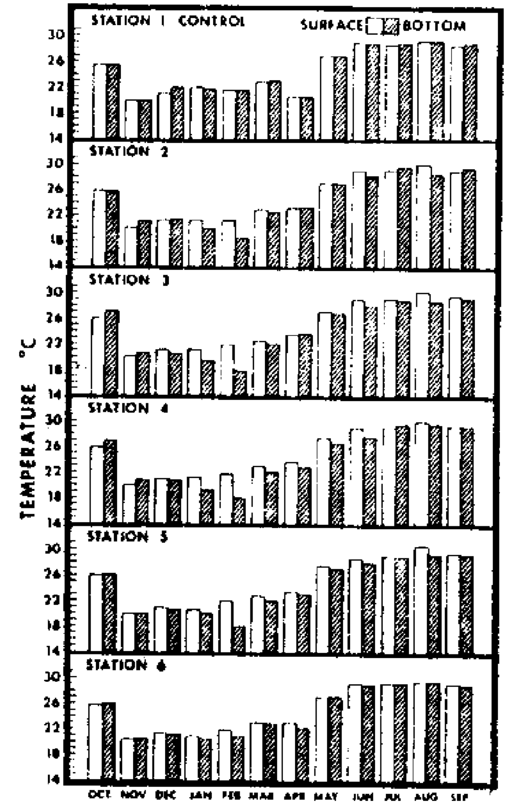


Figure 2.19b - Monthly Water Temperature at the Surface and Bottom of all Hydrologic Stations, October, 1971 - September, 1972 (Source: Lindal, Fable, and Collins, 1975, p. 82).

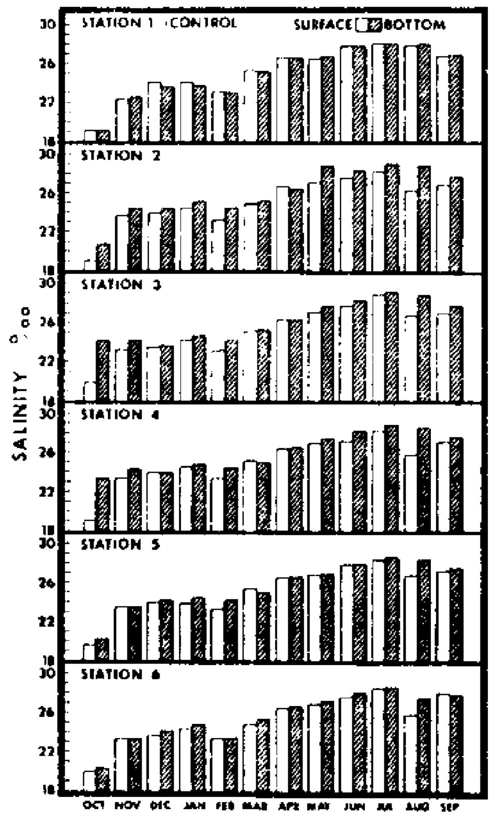


Figure 2.19c - Monthly Salinity at the Surface and Bottom of all Hydrologic Stations, October, 1971 - September 1972 (Source: Lindall, Fable, and Collins, 1975, p. 83).

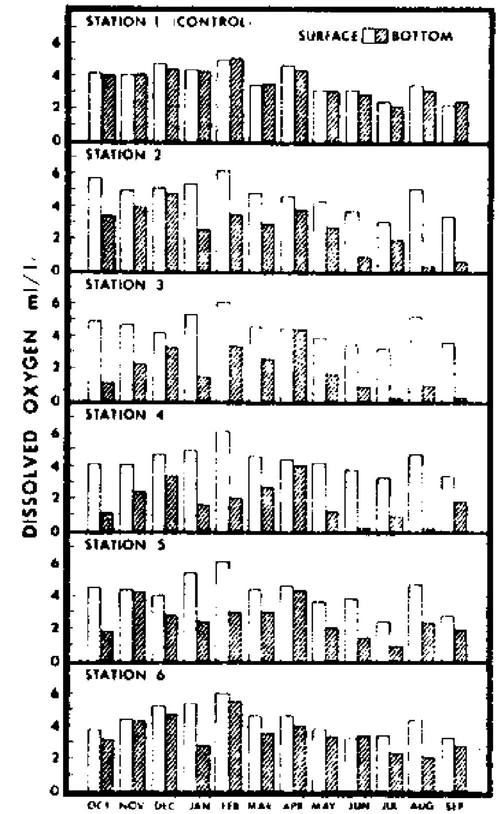


Figure 2.19d - Monthly Dissolved Oxygen at the Surface and Bottom of all Hydrologic Stations, October, 1971 - September 1972 (Source: Lindall, Fable, and Collins, 1975, p. 83).

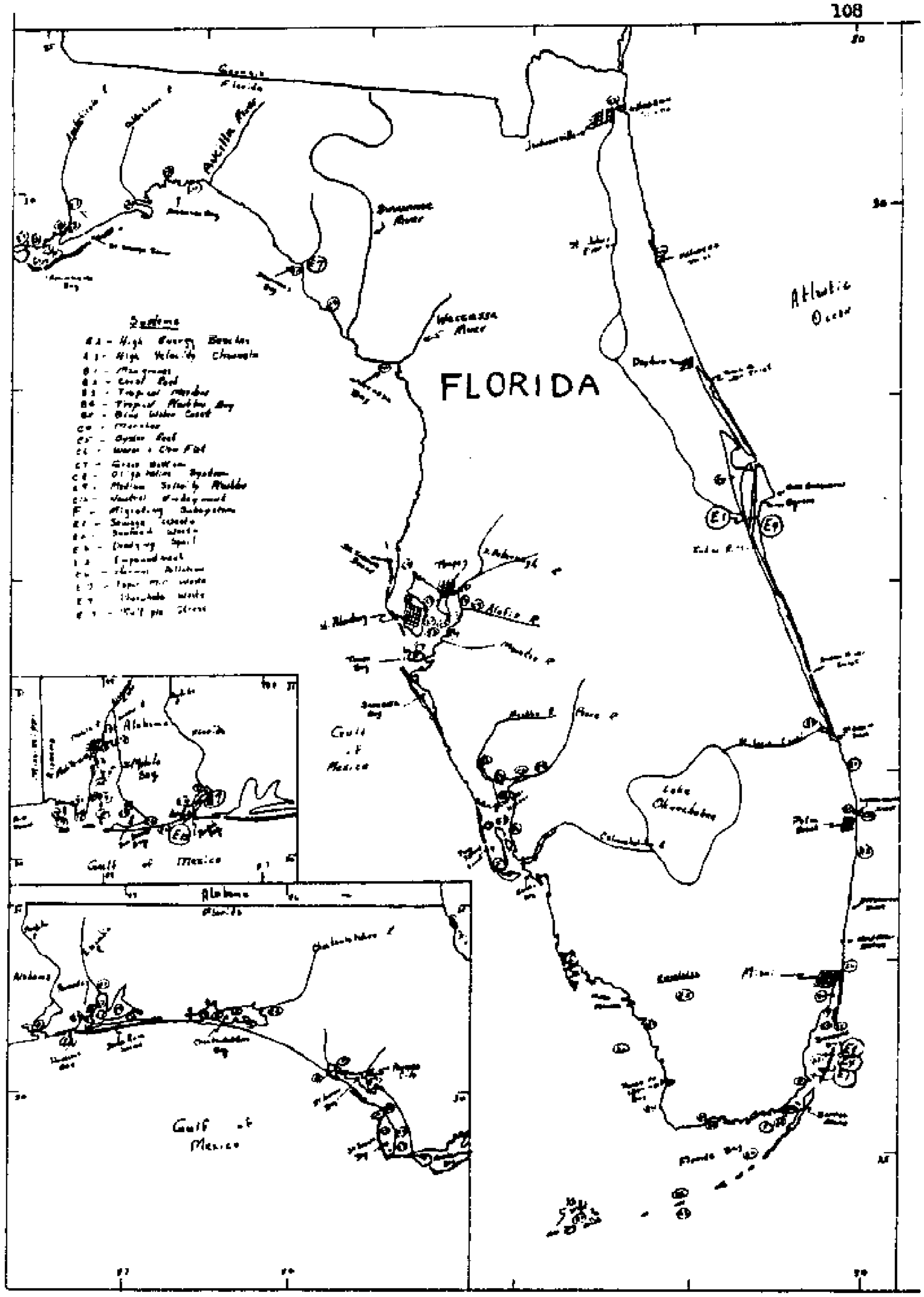


Figure 2.20 - Location map of some examples of coastal ecological systems in Florida (Source: Odum, Copeland and McMahan , 1974, p. 108).

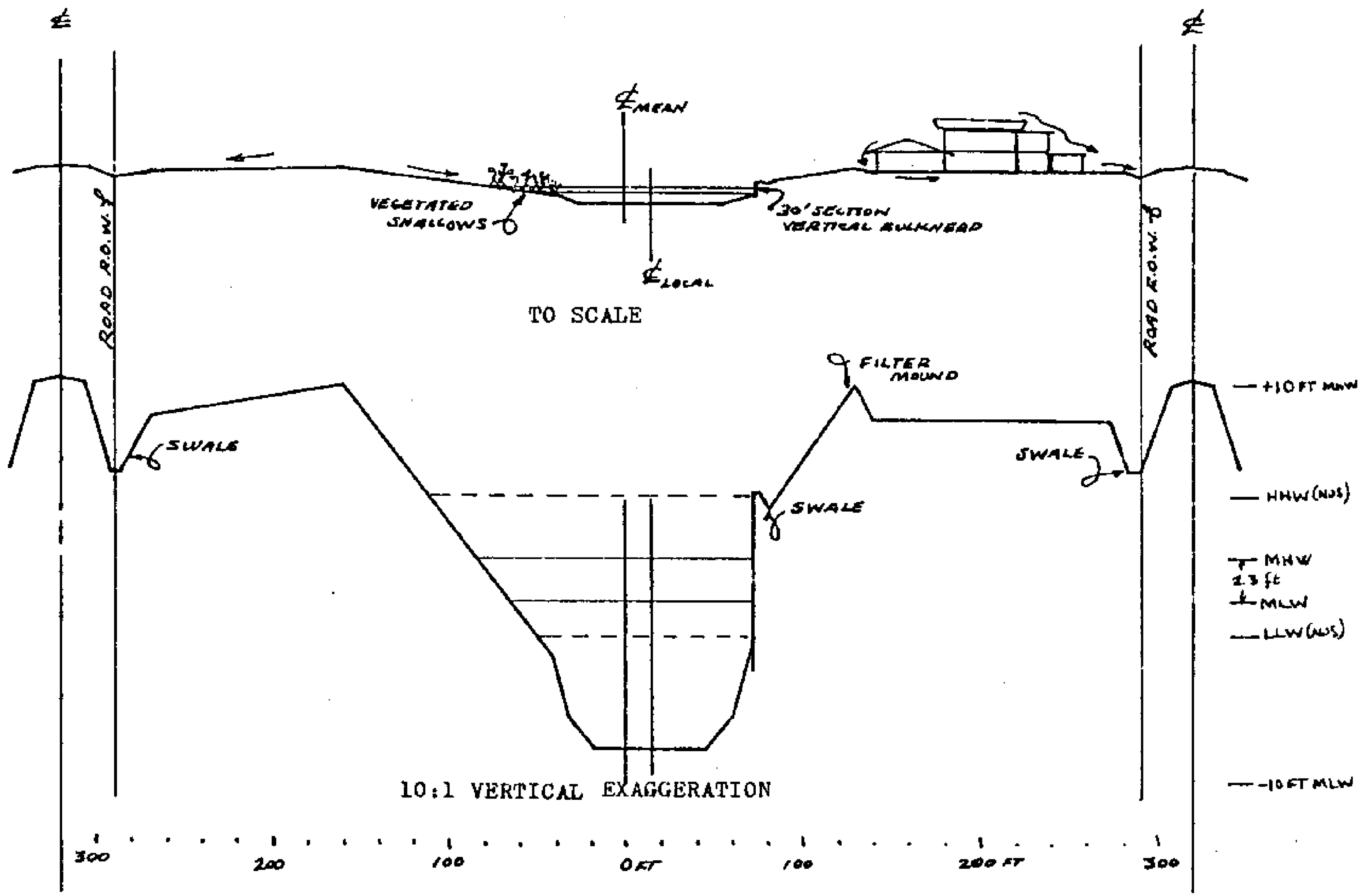


Figure 2.21 - Typical Bank Section Across Canal Showing Filter Mound and Swale, From 57 Acres Canal Design (Source: Snyder, 1976, p. 78).

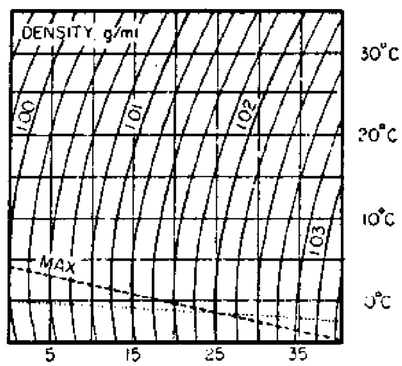


Figure 2.22 - Relationship of Density of Seawater (g/ml) to Salinity (ppt) and temperature (°C). (Source: Chow, 1964, p. 2-5)

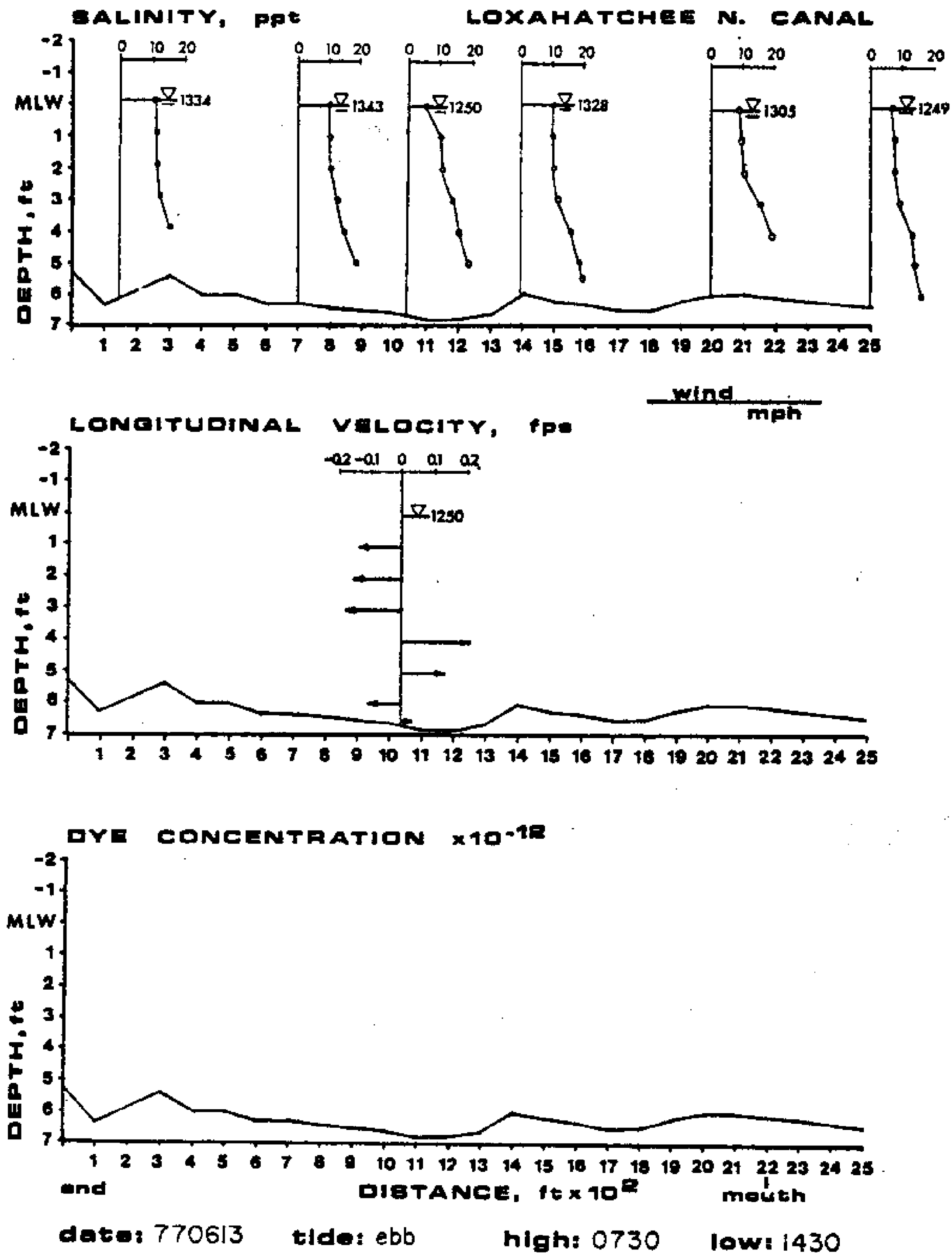


Figure 2.23 - Vertical Salinity Profiles, Loxahatchee North Canal, Showing Presence of Density Wedge [Date: 770613; Time: 1249-1334]

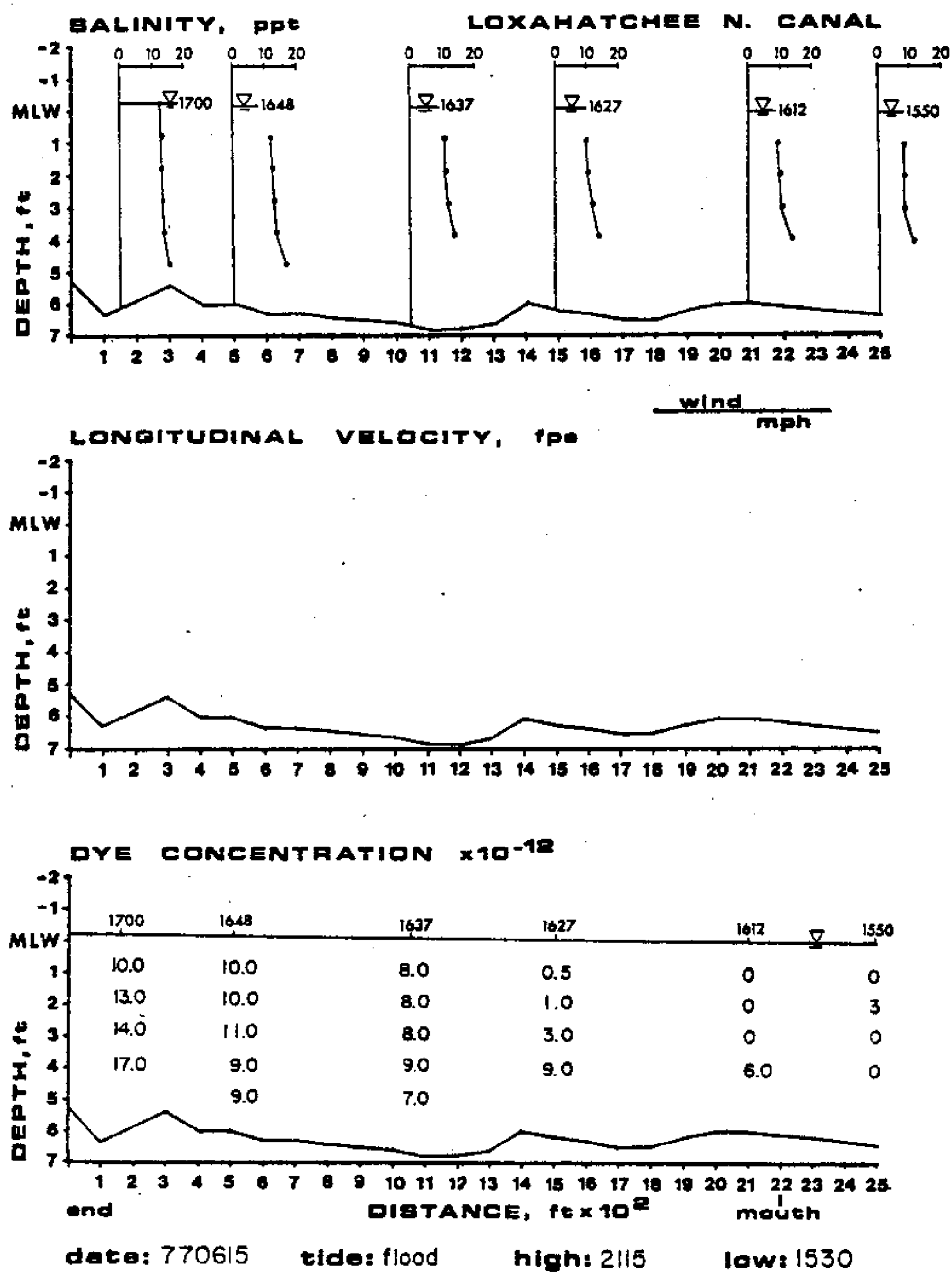


Figure 2.24 - Vertical Salinity Profiles, Loxahatchee North Canal, Showing Slight Density Gradient [Date: 770615; Time: 1550-1700]

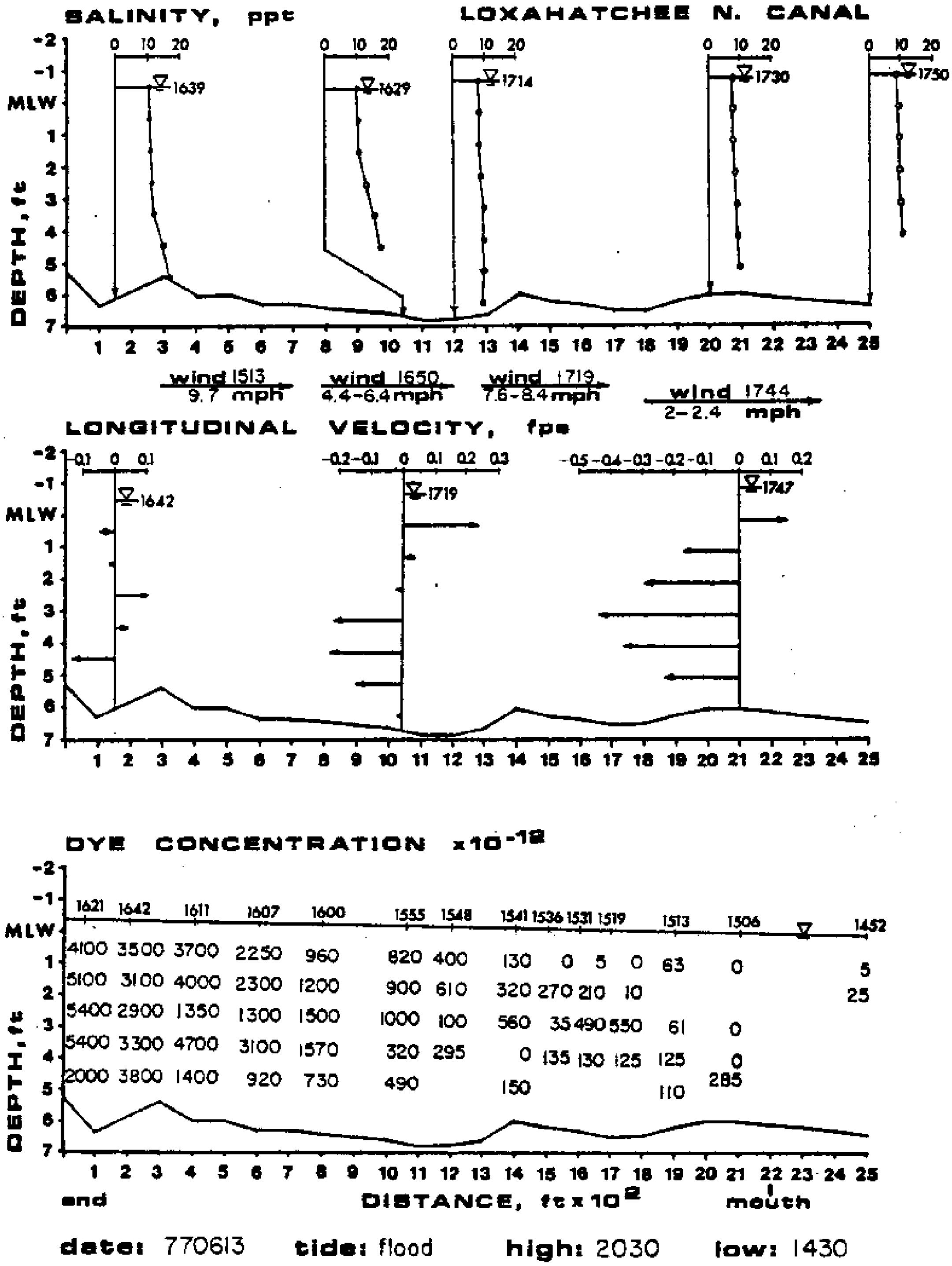


Figure 2.25 - Vertical Salinity Profiles, Loxahatchee North Canal Showing Complete Mixing in the Half Near the Mouth [Date: 770613; Time: 1639-1750]

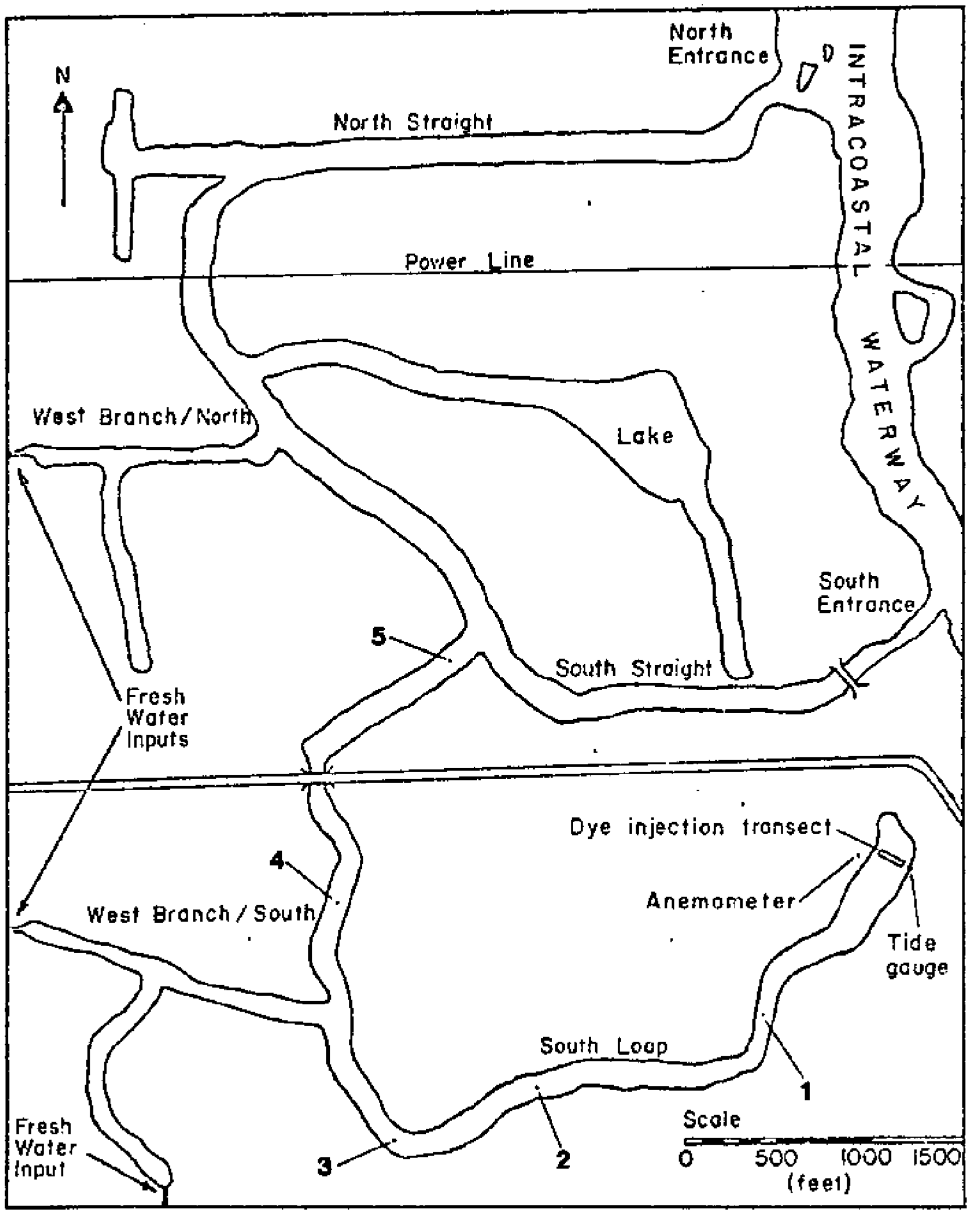


Figure 2.26 - 57 Acres Site Plan Showing Location of Electromagnetic Current Meters for October 1977 Velocity, Salinity, and Water Temperature Measurements.

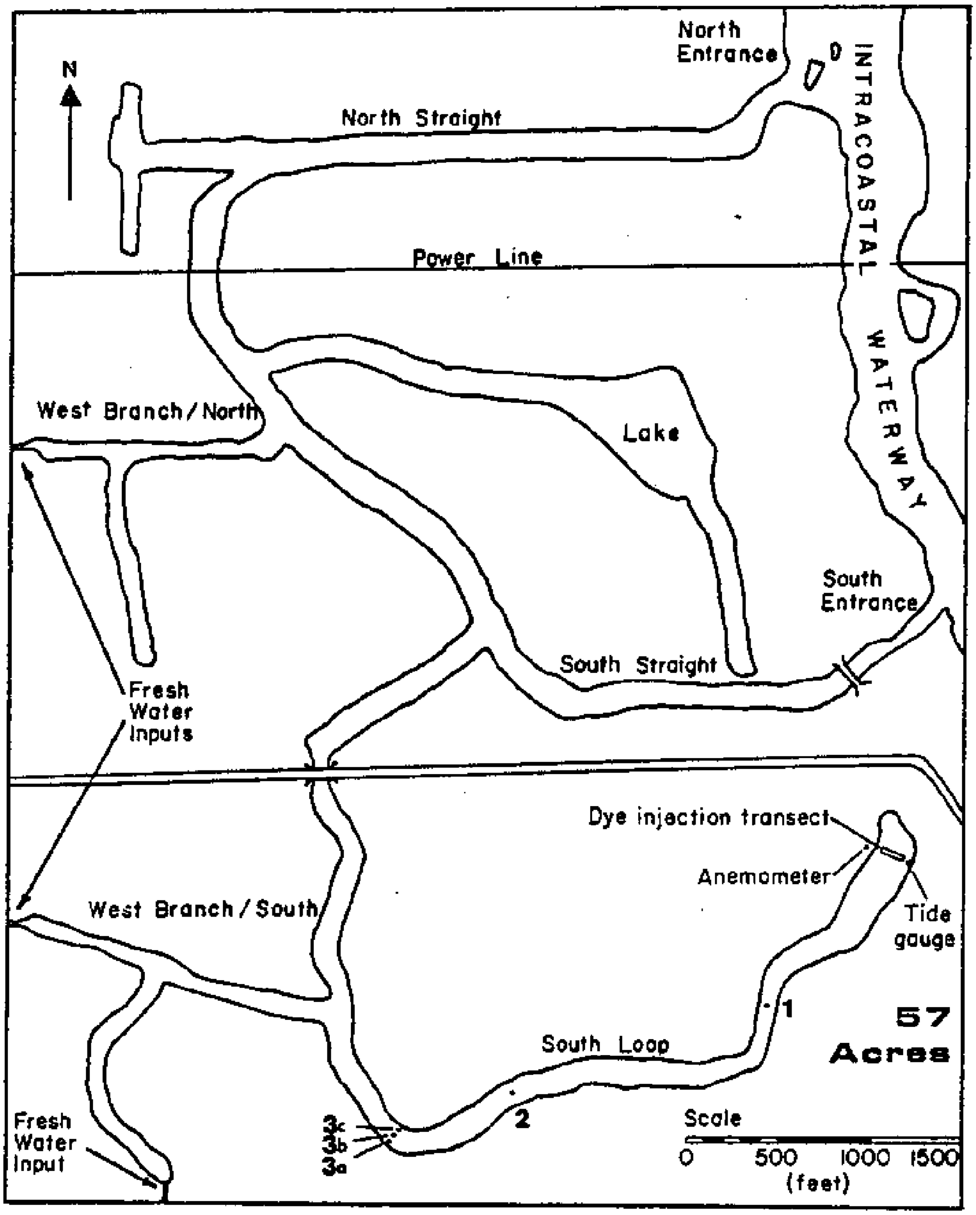


Figure 2.27 - 57 Acres Site Plan Showing Location of Electromagnetic Current Meters for October 1977 Dye Dispersion Measurements.

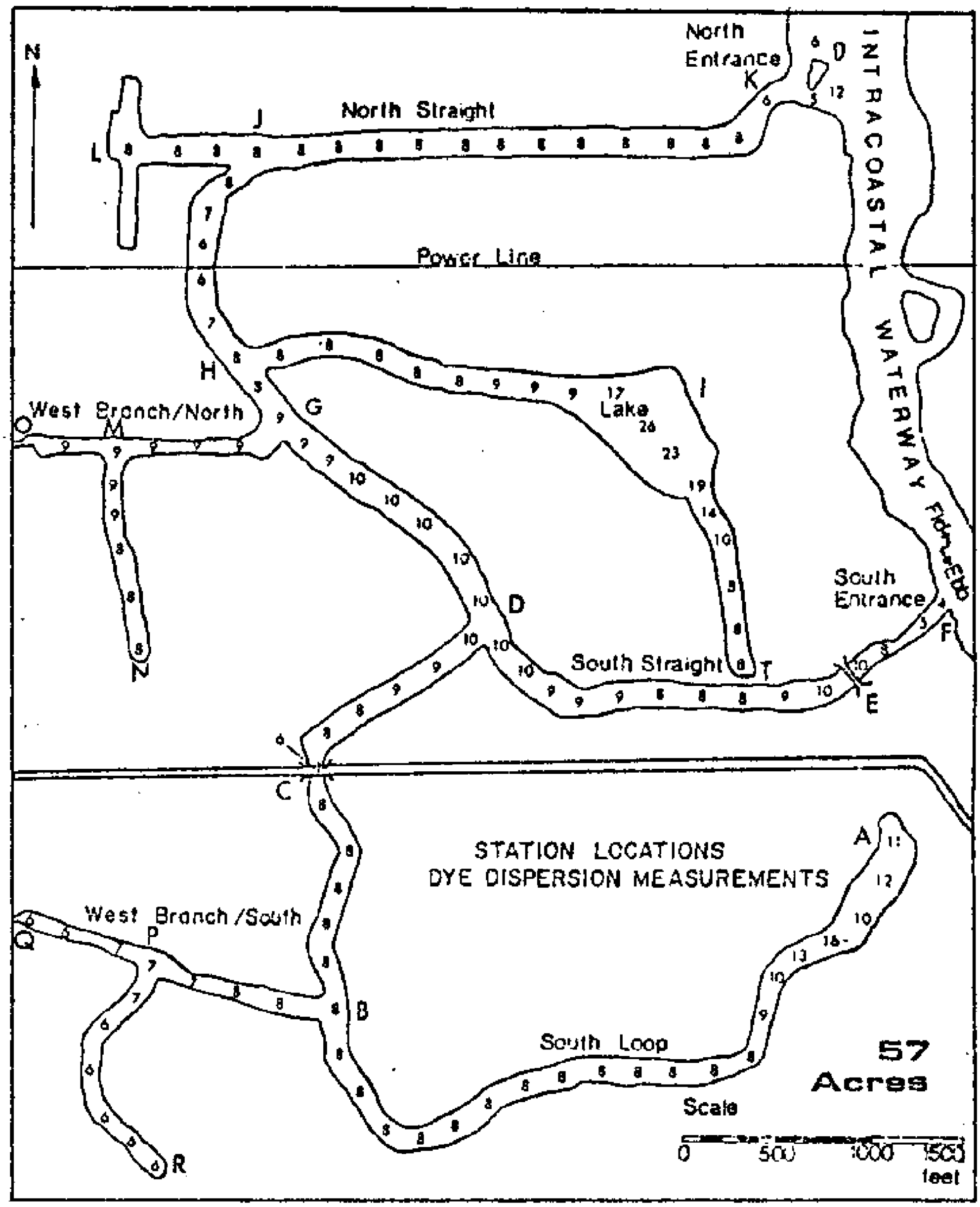


Figure 2.28 - 57 Acres Site Plan Showing Locations of Stations Used to Designate Reaches for October 1977 Dye Dispersion Measurements

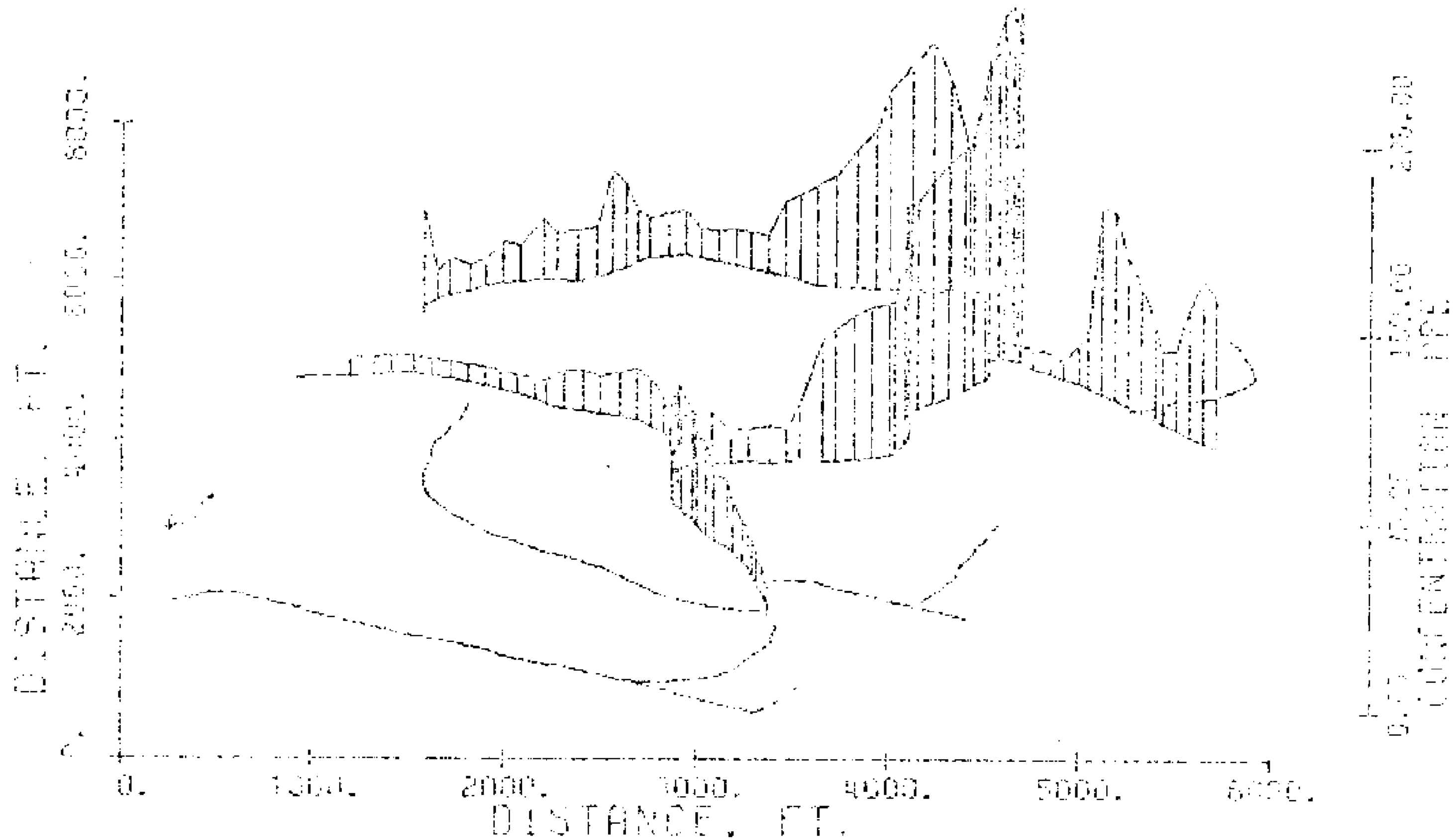


Figure 2.29 - Measured Dye Concentration at 3 ft Depth Along Centerline.
57 Acres Canal System, Palm Beach County, FL
Date: 770720 Time: 1935

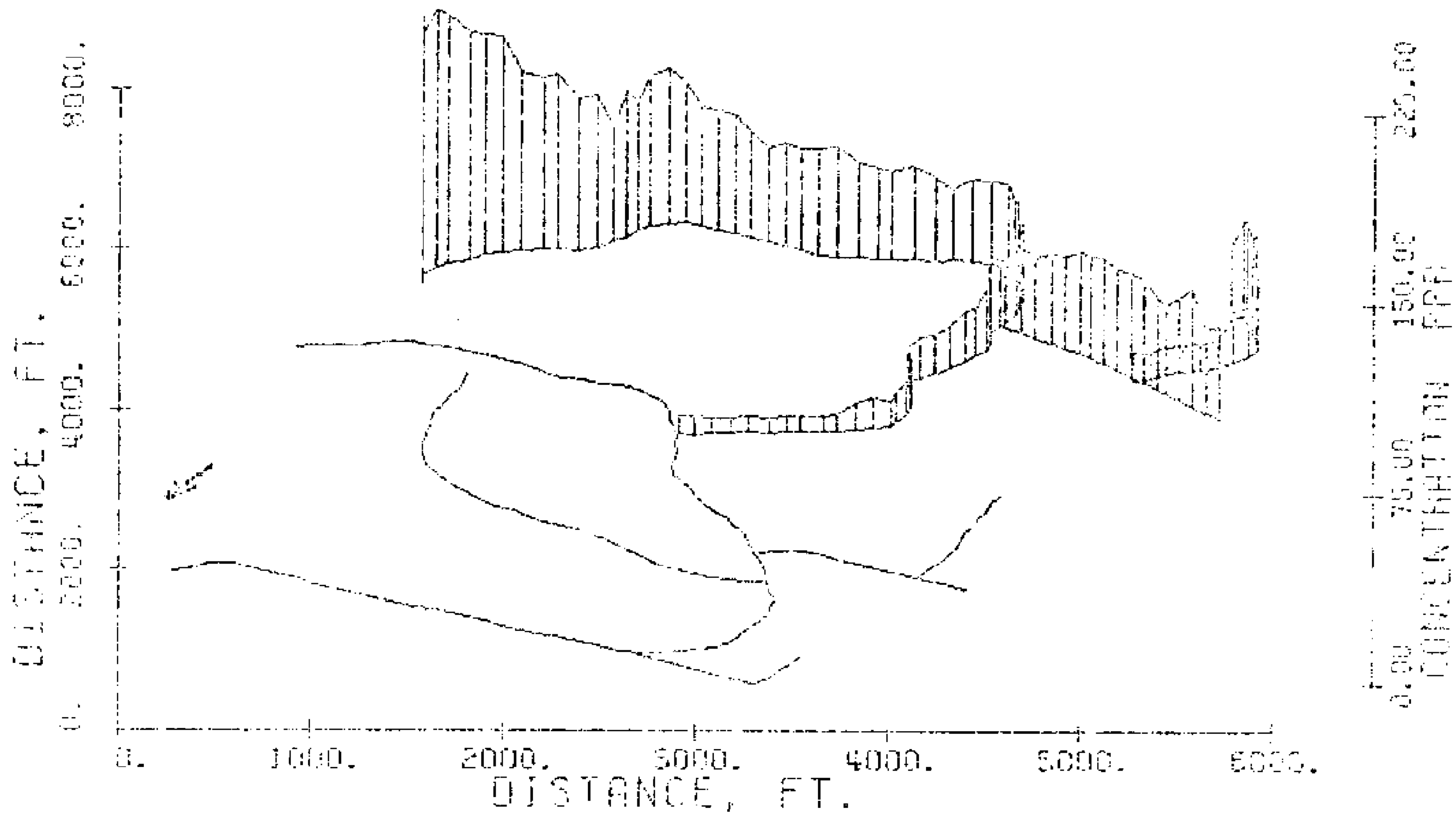


Figure 2.30 - Measured Dye Concentration at 3 ft Depth Along Centerline
57 Acres Canal System, Palm Beach County, FL
Date: 770721 Time: 0105

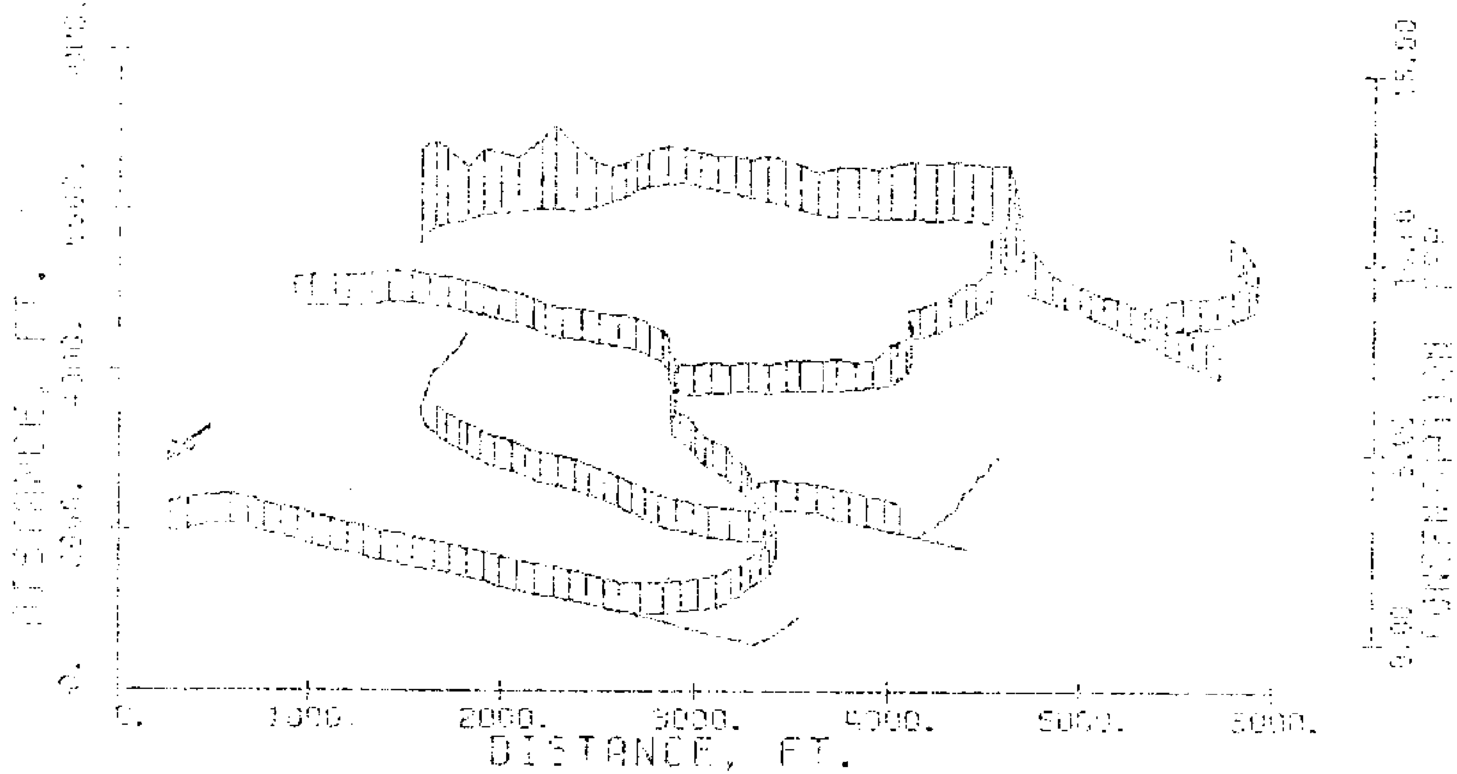


Figure 2.31 - Measured Dye Concentration at 3 ft Depth Along Centerline
57 Acres Canal System, Palm Beach County, FL
Date: 771021 Time: 0420

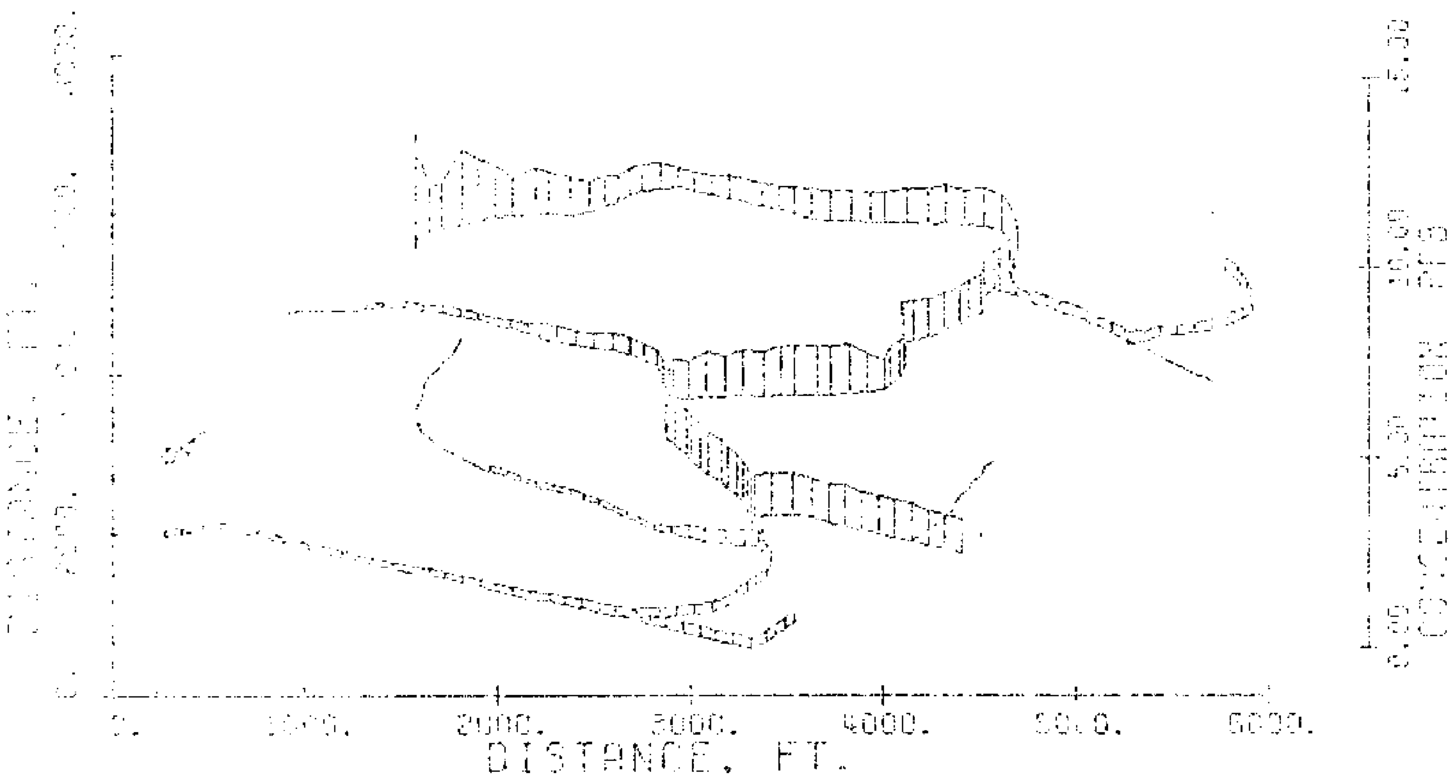


Figure 2.32 - Measured Dye Concentration at 3 ft Depth Along Centerline
57 Acres Canal System, Palm Beach County, FL
Date: 771021 Time: 1045

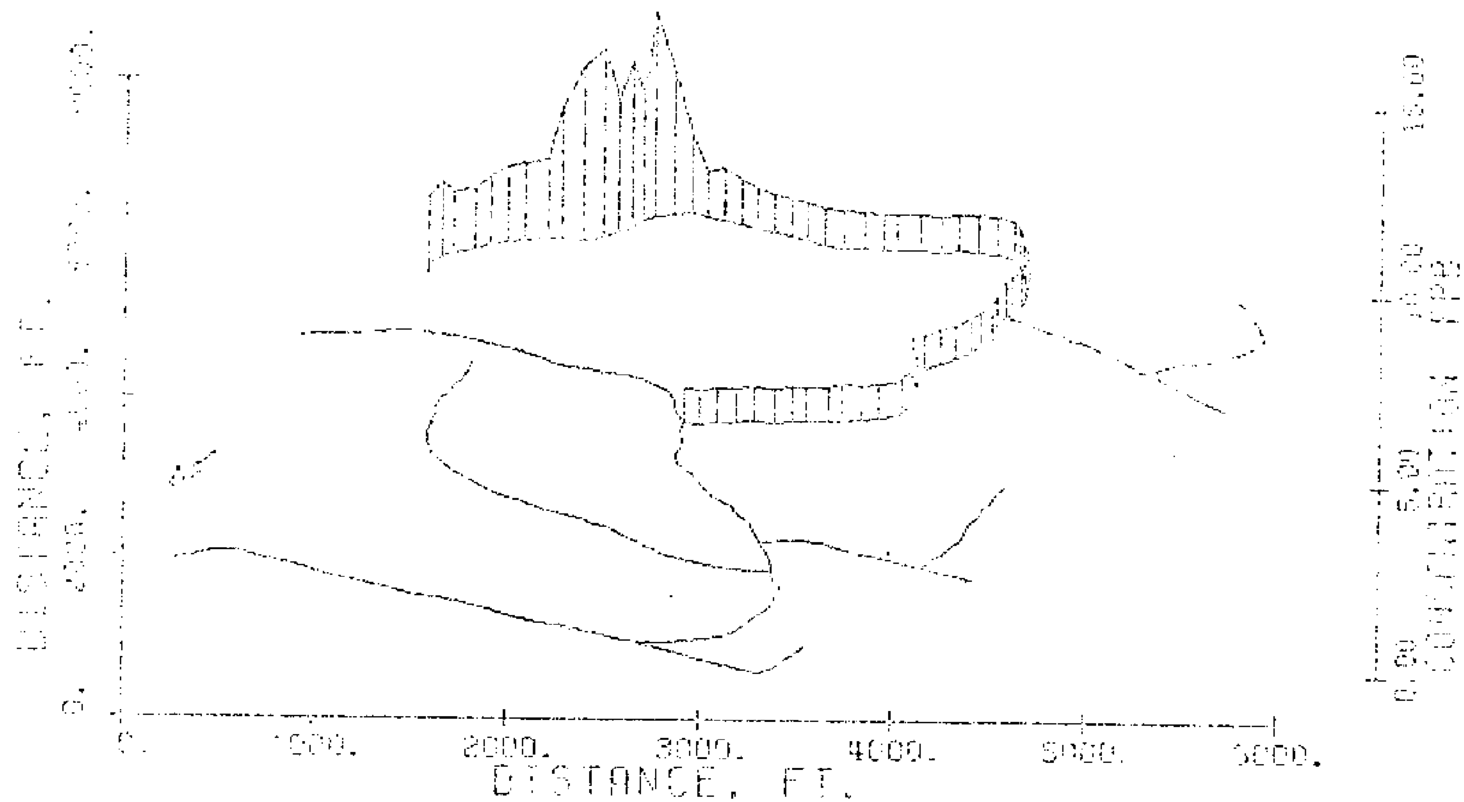


Figure 2.33 - Measured Dye Concentration at 3 ft Depth Along Centerline
57 Acres Canal System, Palm Beach County, FL
Date: 771020 Time: 0330

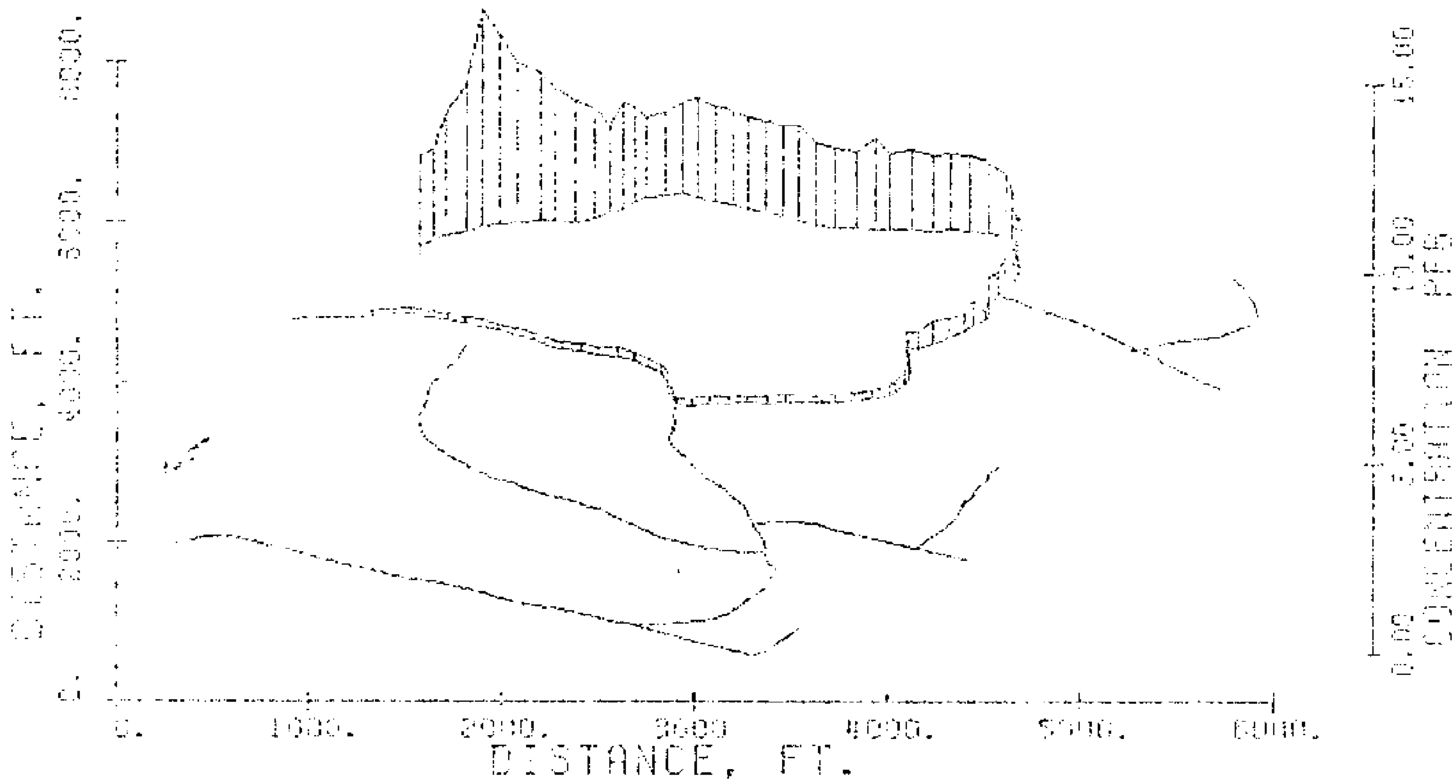


Figure 2.34 - Measured Dye Concentration at 3 ft Depth Along Centerline
57 Acres Canal System, Palm Beach County, FL
Date: 771020 Time: 0950

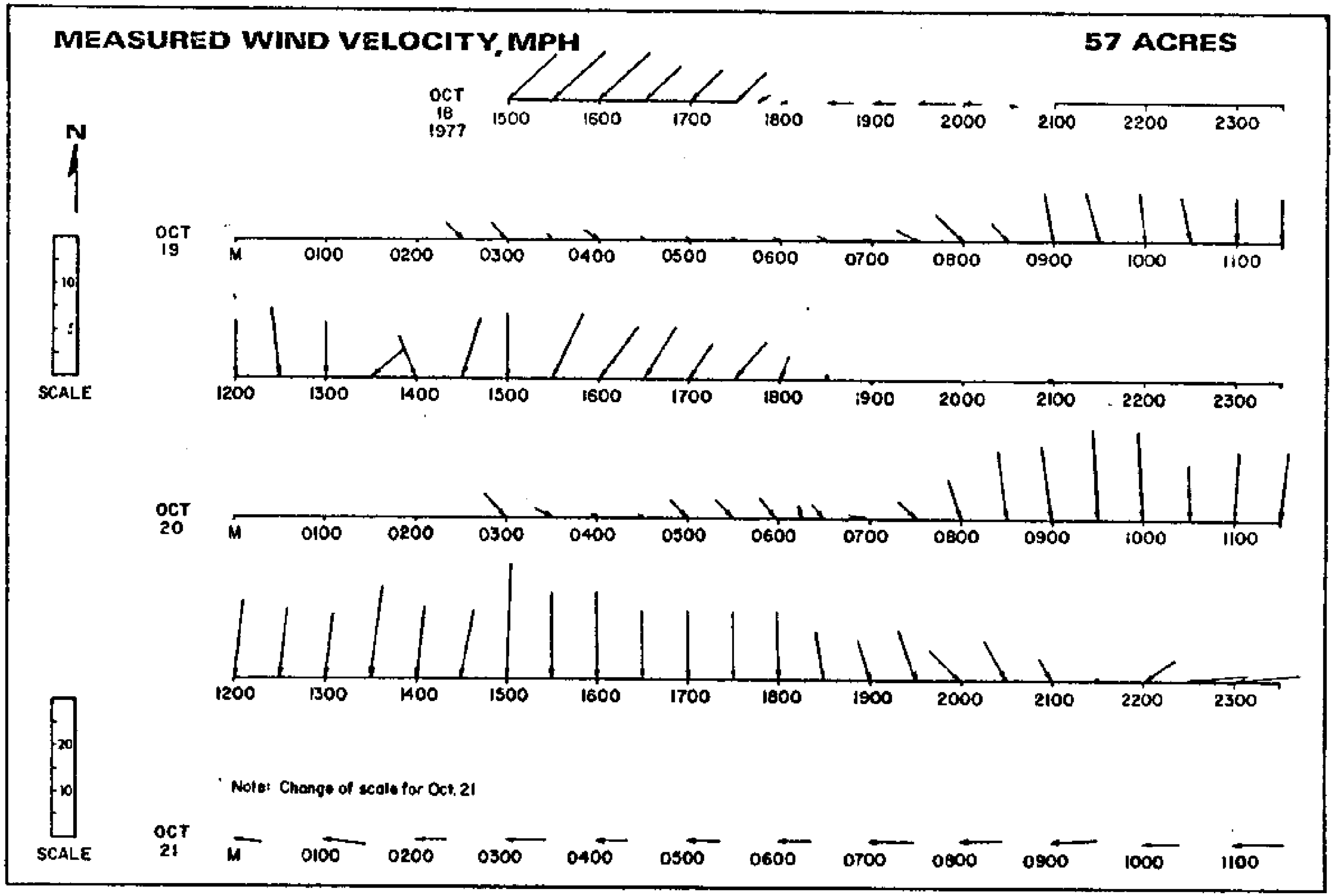


Figure 2.35 - Measured Wind Velocity, 57 Acres Canal System, October 1977

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, 1972, 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 - 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 Charolotte 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 2.3 - 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		
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 2.3 - continued.

Location	Dates	Type	Tide	Depth	Current Speed	Current Velocity	Wind Speed	Wind Velocity	Salinity	Temp.	DO	Dispersion	Bed and Bank Material
D. Detailed Circulation Studies													
57 Acres	760401-760404	FS	X			X	X						
	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 2.4 - 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 $K = E_\ell/Ru_*$
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	N	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	6.9	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

LEGEND:
TIDE: E = Ebb
F = Flood

Source: Morris and Christensen, 1976a, p. 87.

CHAPTER 3

ANALYSIS OF NUMERICAL MODELING TECHNIQUES

3.1 Development of One-Dimensional Model

Before the development of a three-dimensional model was begun, it was decided to develop a one-dimensional mass-transport model to predict the transient concentration profiles of a passive, conservative substance in a simple canal network. A number of numerical techniques were tested and the results analysed for accuracy, stability and transportiveness. The various techniques used are discussed in the next five sections of this chapter. In Section 3.7 the techniques are compared, their results analysed, and a method selected for extension to a three-dimensional model.

In the remainder of this section, the geometry, hydrodynamics, transport equation and dispersion coefficients, and boundary conditions for the one-dimensional model are developed.

3.1.1 Classification and Definitions of Canal Network Geometry

The majority of coastal finger canal networks in Florida consist of straight, prismatic reaches with trapezoidal cross-sections and relatively uniform depths, separated by junctions (see Figures 1.1 - 1.3). At the *tidal entrances*, a canal network has one or more hydraulic connections with a receiving waterbody, usually either the Intracoastal Waterway (ICW) or the ocean, or in some cases (particularly on the Gulf Coast) to an estuary. From the tidal entrances, a canal network consists of an upward branching system of canals, possibly with loops, culminating in what will be called dead-ends. The *dead-ends* of a canal network are the upward limits of the system at which there is either an impervious boundary, a freshwater flow over a salinity control structure, or else at a distance sufficiently far up a river that the hydrodynamic effects of tides are negligible. A *loop* is defined as that part of a canal network in which a closed continuous line can be drawn along the longitudinal center lines of component reaches. The area of the water surface associated with each loop includes not only the surface areas of the reaches and junctions joined by the continuous line, but also the surface areas of canals and junctions which are hydraulically further away from that point of the continuous line which has the shortest hydraulic connection to a tidal entrance, and which have hydraulic connections to reaches and junctions along the continuous line (see Figure 3.1).

In this report, the positive x-axis for each component reach will be defined from the point of the reach which is hydraulically farthest away from the tidal entrances, along the longitudinal center line in the direction of the ebb tide (or towards the point closest to the tidal entrances). This will also be termed the *downstream direction*. Similarly the *upstream direction* will be defined along the negative x-axis. Facing downstream, the *left bank* will be the bank to the left of the longitudinal center line of the reach, and the *right bank* will be the bank to the right.

At a junction, which is the meeting point of two to four distinct reaches, the *downstream reach* for that junction is defined as the reach which lies hydraulically closer to the tidal entrance.

One of the remaining reaches coming together at the junction is then defined to be the *upstream reach* for that junction. When four reaches meet at a junction, this is the middle of the three upstream reaches. When only three reaches meet, the upstream reach is designated as that which is a more continuous extension of the downstream reach in the modeler's judgement. Once the upstream and downstream reaches are defined, the positive x-axis through the junction is from the former to the latter. Facing in this direction, a canal joining the junction from the left is termed the *left branch* for that junction, and a canal joining from the right is called the *right branch*.

All the above definitions are embodied in Figure 3.1.

3.1.2 Model Canal Network Geometry

To analyse some of the various numerical solution techniques available, two simple canal networks were defined. Each reach of the two networks was assumed to be trapezoidal in cross-section with bottom width, b, and with the same inverse side slope, s, on each side of the canal (Figure 3.2). The cross-sectional area, A, for any depth of water, d, is then given by,

$$A = d(b + sd) \tag{3.1}$$

The first network (Figure 3.3) consisted of a single straight prismatic canal of length, L, with a dead-end at the upstream end and a tidal entrance downstream. The second network (Figure 3.4) consisted of two canals meeting at a junction with a single tidal entrance at the downstream end, dividing the system into three reaches of lengths L_i, where i refers to the number of the reach. In Figures 3.3 and 3.4 the reach number is the uncircled number and the junction number the circled number. For computational purposes, it was easier to refer to each dead-end as junction number 1, to number interior junctions consecutively from 2 to (NJUNC + 1), where NJUNC is the number of interior junctions, and to number tidal entrances consecutively from (NJUNC + 2) upward.

3.1.3 Hydrodynamics of One-Dimensional System

There are several mechanisms which contribute to the hydodynamics of canal networks, and these can be classified into two groups which have an internal or an external effect on the system. The internal mechanisms are those which cause a variation within the canal network by acting directly on the interior of the system. The external mechanisms are those which operate on the receiving waterbody, and which transfer their effects through the tidal entrances.

Examples of internal mechanisms are lateral inflow, secondary currents, wind induced circulation and density driven currents, induced by a saline wedge entering the canal network during the flood tide and retreating during the ebb tide, or by thermal gradients. However, all these effects except lateral inflow, produce circulation patterns whose net mass flux through a cross-section is zero at any time, and hence which cannot be modeled using a one-dimensional numerical scheme. To compensate, many modelers try to reproduce the three-dimensional effects by adjusting the one-dimensional model's coefficients, but the results are rarely satisfactory as the circulation pattern is not followed accurately enough. Only the lateral inflow, of the above list, can be thus modeled, because it has positive net flux in the system.

The main external influences are the astronomical tides, baroclinically induced tides and wind tides. All these phenomena have the effect of raising or lowering the water elevation at the tidal entrances to the canal network, and thus directly inducing a response within the system. In the one-dimensional model, these effects can be modeled in one of two ways. If the record of tidal elevations at the tidal entrances to the canal network follows a simple harmonic function, then the amplitude and frequency (or period) of this distribution can be read into the model, and the tidal elevations generated. Otherwise, the record can be digitized and read directly into the model as an input tidal entrance boundary condition.

Most hydrodynamic models are based on the de Saint-Venant Equations of continuity, and momentum (or the dynamic equation as it is frequently termed). In one such model [Harleman and Lee, 1969], which will be used as a comparison with the hydrodynamic model presented in this section, the equations are written as follows, continuity,

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} - q_I = 0 \tag{3.2}$$

and momentum,

$$\frac{\partial Q}{\partial t} + u\,\frac{\partial Q}{\partial x} + Q\,\frac{\partial u}{\partial x} + g\,\frac{\partial d}{\partial x}\,A + g\,\frac{Q|Q|}{AC^2R} = 0 \tag{3.3}$$

where

t = time, (T)

Q = discharge, (L^3/T)

x = longitudinal displacement, (L)

q_I = lateral inflow per unit length of reach, (L^2/T)

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

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

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

R = hydraulic radius, (L)

This model was used to study the hydrodynamics of a single canal with a variety of end boundary conditions. However, a number of investigators have used similar models to study canal networks with multiple branches, tidal entrances and loops [Vreugdenhil, 1973; Abbott, et al, 1975].

A study of the hydrodynamic variables measured in some of Florida's coastal finger canal networks indicates that it is usually not necessary to develop a one-dimensional hydrodynamic model based on the full set of equations. Canal networks in Florida are mostly fairly short, less than 10,000 ft long for example. They usually consist of straight prismatic reaches with negligible bed slopes separated by junctions. Typically, tidal ranges on the Atlantic coast are about 2 to 4 ft (Figure 3.5), and along the Gulf coast are 2 to 3 ft (Figure 3.6) [NOAA, 1975]. These small tidal ranges induce very small velocities in the networks (Figure 3.7), and is not uncommon to measure maximum velocities of less than 0.5 ft/sec at the tidal entrances (Figure 3.7). These and other typically measured hydrodynamic parameters are listed in Table 3.1. As can be seen from this table, maximum measured water surface slopes have been found to be between 10^{-5} and 10^{-6}.

This last observation in particular led Christensen [1975], to suggest that the acceleration terms in the momentum Equation (3.3) might be negligible in this case, and that a good approximation to the flow would be to consider a horizontal water surface rising and falling with the tidal frequency and amplitude. Thus, the continuity equation alone would be sufficient to uniquely determine the velocity field [Walton, 1976a]. To test this assumption, a simple hydrodynamic model based on the conservation of mass principle was developed for the first test canal network (Figure 3.3), and the results compared with those obtained from Harleman and Lee's model for the same system.

For all the one-dimensional modeling, it was assumed that the depth varied sinusoidally with the tidal amplitude, a, and frequency, ω, (Figure 3.8). Thus the depth, d_i, in any reach, i, of the canal network, is given by,

$$d_i = d_{o_i} + a \cos \omega t \tag{3.4}$$

where

d_{o_i} = mean tidal depth in reach i, (L)

and

$$\omega = \frac{2\pi}{T} \tag{3.5}$$

where

T = tidal period = 12.42 hrs.

Such a representation is consistent with tides on the Atlantic coast Figure (3.5).

As the discharge for reach i, Q_i, can be written

$$Q_i = A_i u_i \tag{3.6}$$

where the area A_i is not a function of x, Equation (3.2) becomes,

$$\frac{dA_i}{dt} + A_i \frac{du_i}{dx} - q_{I_i} = 0 \tag{3.7}$$

Integrating with respect to x_i from the upstream section of the reach,

$$u_i(x,t) = (\int_o^{x_i} q_{I_i}\, dx - x_i \frac{dA_i}{dt})/A_i + u_{o_i} \tag{3.8}$$

where

u_{o_i} = velocity at upstream section of the reach, (L/T).

The velocity, u_{o_i}, can be calculated in a similar manner in the upward branches of the reach considered, giving,

$$u_i(x,t) = [\sum_j^{Nu} (\int_o^{L_j} q_{I_j}\, dx - L_j \frac{dA_j}{dt}) + \int_o^{x_i} q_{I_i}\, dx - x_i \frac{dA_i}{dt}]/A_i \tag{3.9}$$

where

Nu = number of upstream reaches.

Now

$$\frac{dA_i}{dt} = B \frac{dd}{dt} \tag{3.10}$$

and

$$A_{ws} = B\ L \tag{3.11}$$

where

A_{ws} = area of water surface in canal network upstream of a section (L^2),

and

B = top width of cross-section, (L),

thus Equation (3.9) can be simplified to,

$$u_i(x,t) = [\sum_j^{Nu} \int_o^{L_j} q_{I_j}\, dx + \int_o^{x_i} q_{I_i}\, dx]/A_i - \frac{A_{ws_i}}{A_i} \frac{dd}{dt} \tag{3.12}$$

Equations (3.4) and(3.12) are then sufficient to determine the depth of flow, and the velocity field at any point in the model one-dimensional canal network as a closed function of x and t.

To test the accuracy of this model, the depths and velocities were computed at various points inside the first test canal, the single prismatic canal (Figure 3.3), and the results compared with the results obtained from Harleman and Lee's model for the same system. Many of the parameters were varied, in particular, the canal length up to 11,000 ft. In all cases the depths and velocities calculated from the two models were within 2 percent of each other. A typical set of comparative results for a 9,500 ft canal is listed in Table 3.2. This was considered to be sufficiently accurate considering the errors inherent in numerical modeling and in the practical measurement of field data. Equations (3.4) and 3.12) were therefore used to describe the hydrodynamics in a one-dimensional canal network.

3.1.4 Transport Equation and Dispersion Coefficient

The three-dimensional mass-transport equation is developed by considering the conservation of a substance in an elemental volume of the flow. Using Fick's first law for molecular diffusion and an analogy for turbulent diffusion, the equation can be written, [Harleman, 1966, pp. 576-578; Pritchard, 1971, p. 16],

$$\frac{\partial c}{\partial t} + \frac{\partial}{\partial x}(cu) + \frac{\partial}{\partial y}(cv) + \frac{\partial}{\partial z}(cw) = \frac{\partial}{\partial x}\left(E_x \frac{\partial c}{\partial x}\right) + \frac{\partial}{\partial y}\left(E_y \frac{\partial c}{\partial y}\right) + \frac{\partial}{\partial z}\left(E_z \frac{\partial c}{\partial z}\right) + r_p \tag{3.13}$$

where

x, y, z = coordinate directions, (L)

u, v, w = velocities in x, y, z directions respectively, (L/T)

E_x, E_y, E_z = turbulent diffusion coefficients in x, y, z directions respectively, (L^2/T)

c = concentration, (dimensionless)

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

The one-dimensional mass transport equation can then be obtained by cross-sectionally averaging Equation (3.13) [Harleman, 1971, pp. 37-38] and by using a Fick's first law analogy for longitudinal dispersion [Taylor, 1954; Aris, 1956],

$$\frac{\partial}{\partial t}(Ac) + \frac{\partial}{\partial x}(Auc) = \frac{\partial}{\partial x}\left(A(E_x + E_\ell)\frac{\partial c}{\partial x}\right) + Ar_p \tag{3.14}$$

where

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

Elder [1959], examined the orders of magnitude of the longitudinal turbulent diffusion coefficient, E_x, and the longitudinal dispersion coefficient, E_ℓ, and found that the latter was about ten times the former. Thus, Equation (3.14) is often written,

$$\frac{\partial}{\partial t}(Ac) + \frac{\partial}{\partial x}(Auc) = \frac{\partial}{\partial x}\left(AE_\ell\frac{\partial c}{\partial x}\right) + Ar_p \tag{3.15}$$

although measurement techniques for the longitudinal dispersion coefficient are integral methods and thus automatically include the effect of the longitudinal diffusion coefficient.

As the substance being modeled was considered to be passive and conservative, and as there were no sinks for the flow except at the tidal entrances, the rate of production term, r_p, includes only the lateral inflow, q_I. If the concentration of the lateral inflow is c_I, then the rate of production is given by,

$$r_p = q_I c_I / A \tag{3.16}$$

and Equation (3.15) becomes

$$\frac{\partial}{\partial t}(Ac) + \frac{\partial}{\partial x}(Auc) = \frac{\partial}{\partial x}(AE_\ell \frac{\partial c}{\partial x}) + q_I\, c_I \tag{3.17}$$

The above equation is called the conservative form of the one-dimensional mass-transport equation. The term *conservative* is used to imply that a finite difference approximation of the equation preserves the integral relationship of the continuity equation [Roache, 1972, pp. 28-33]. A *non-conservative* form of the equation does not have this property. Roache states that a conservative formulation does not always give more accurate results, but that the experience of many researchers has indicated that such formulations generally are more accurate. A non-conservative form of Equation (3.17), using the continuity Equation (3.3) and the fact that the area, A, is not a function of the longitudinal displacement, is,

$$\frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} = \frac{\partial}{\partial x}(E_\ell \frac{\partial c}{\partial x}) + q_I\, (c_I - c)/A \tag{3.18}$$

It will be seen later that it is sometimes necessary to use this form of the equation.

Classically, the longitudinal dispersion coefficient, E_ℓ, is written [Taylor, 1954; Elder, 1959],

$$E_\ell = KRu^* \tag{3.19}$$

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 system, depending on radii of bends, bottom roughness, wall Reynolds' number, etc. Although classically considered to be about twenty (based on the hydraulic radius), many researchers have measured values of K in various types of waterbodies, from less than ten to the order of several hundreds.

In many Floridian canals, logarithmic velocity profiles have been observed when the effects of wind induced circulation and density driven currents have been small (Figure 3.7). Such a profile gives a unique relationship between the spatial mean velocity, u, and the bed shear velocity, u*, [Nikuradse, 1933],

$$u = 2.5u^* \ln(10.9d/k) \tag{3.20}$$

where

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

Thus, from Equation (3.19)

$$E_\ell = \frac{0.4KRu}{\ln(10.9d/k)} \tag{3.21}$$

As the spatial mean velocity, u, is a closed form function of the longitudinal displacement and time, the longitudinal dispersion coefficient is also a closed form function of x and t. Therefore, the only remaining unknown in the one-dimensional mass-transport Equation (3.17) or (3.18) is the concentration, c.

3.1.5 Boundary Conditions

The one-dimensional mass-transport Equation (3.17) or (3.18) is a second-order parabolic equation, being second-order in the spatial variable, x, and first-order in the time variable, t. Thus, to close the system, two spatial boundary conditions and a set of initial conditions are required.

For both the test canal networks, the initial conditions were that the concentrations at time t = 0 in the canal either equaled the background concentration, c_{RW}, or else that they were read in.

At the dead-ends of the canal system, a zero-flux condition was initially used in the form,

$$\frac{\partial c}{\partial x} = 0 \tag{3.22}$$

However, it soon became apparent that simple numerical approximations to Equation (3.22), such as a forward difference operator,

$$\frac{c_1 - c_o}{\Delta x} = 0 \tag{3.23}$$

where

c_o = concentration at dead-end, (dimensionless)

c_1 = concentration at adjacent node, (dimensionless)

implied that the gradient of the concentration profile in the segment of the reach adjacent to the dead-end is zero, which led to erroneous results. Higher-order schemes were discounted because it was felt that they used information which was too far away from the dead-end to be meaningful.

To overcome this problem, a simple mass balance expression was developed based on the mean velocity between the two nodes at the dead-end. However, this condition was eventually simplified when it was observed that the concentration profiles resulting from this model were linear at low tide, and that the mass balance condition maintained this linearity all the way to the dead-end. Thus, the final form of the dead-end boundary condition for this test model was that the concentration at the dead-end was a linear extrapolation of the concentrations at the two-adjacent nodes,

$$c_o = 2c_1 - c_2 \tag{3.24}$$

At the tidal entrances to the system, a dual condition was established. During the ebb tide a "floating" type boundary condition was used in which the concentration at the entrance was calculated using a backward difference operator. Dispersion was neglected at this point because the concentration would either be convected out of the system at the next time step, or else would be lost in the description of the flood tide boundary condition. Thus during the ebb tide, the concentration at the tidal entrance, c_{TE}^{n+1}, was given from the equation

$$\frac{c_{TE}^{n+1} - c_{TE}^{n}}{\Delta t} = \frac{u_{TE}^{n}\, c_{TE}^{n} - u_{TE-1}^{n}\, c_{TE-1}^{n}}{\Delta x} \tag{3.25}$$

where

Δt = time increment, (T)

Δx = spatial increment, (L)

n = time level

Assuming that the concentration at the tidal entrance reaches a value of c_{LT} at low tide, a first order decay was used to describe the time history of the concentration at the tidal entrance during the flood tide,

$$c_{TE} = c_{RW} + (c_{LT} - c_{RW}) \exp(-3t'/\tau) \tag{3.26}$$

where

t' = time since low tide, (T)

τ = time decay coefficient, (1/T).

The form of this boundary condition results in the concentration at the tidal entrance being within 2 percent of the background concentration, c_{RW}, after τ units of time after low tide.

To test the accuracy of this assumption, a simple two-dimensional finite-element model was developed to describe the transport in a receiving waterbody such as the ICW under a point source/ sink loading, Results indicated that a first order decay type condition was a fair approximation to the resulting flood tide concentration profile at the tidal entrance. Furthermore, as will be seen later in Section 3.8, variation of the time decay coefficient, τ, does not dramatically alter the resulting concentration profiles in the canal network.

3.2 Finite-Difference and Finite-Element Approaches

Measured longitudinal dispersion coefficients in Floridian canals are usually very small, less than 5 ft^2/sec being common. This means that the dispersion term in the one-dimensional mass transport Equation (3.17) or (3.18) is small. If this term is dropped altogether, then the remaining first-order hyperbolic equation is the one-dimensional convection equation,

$$\frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} = 0 \tag{3.27}$$

A number of people have studied this equation [von Neumann and Richtmyer, 1950; Stone and Brian, 1963], and have concluded that model studies of this equation using the more common low order finite-difference approximations usually lead to inaccurate results, and possibly instabilities, depending on the scheme used. A survey of the more common finite-difference schemes such as the forward, backward, central explicit or implicit schemes, will not be given here as they are extensively covered in the literature [Ames, 1969; Milne, 1970; Roache, 1972; Smith, 1975].

The problem, termed numerical dispersion, arises out of a Taylor series approximation to the terms of Equation (3.27). If, for example, a constant velocity, u, is assumed in the positive x-direction and a backward difference scheme is used to approximate the equation to the second order [Molenkamp, 1968],

$$\frac{\partial c}{\partial t} + u\,\frac{\partial c}{\partial x} = \frac{1}{2}\,|u|\,\left(1 - |u|\,\frac{\Delta t}{\Delta x}\right)\,\Delta x\,\frac{\partial^2 c}{\partial x^2} \tag{3.28}$$

the term on the right hand side of Equation (3.28) is the numerical dispersion term with an associated numerical dispersion coefficient

$$E_n = \frac{1}{2}\,|u|\,\left(1 - |u|\,\frac{\Delta t}{\Delta x}\right)\,\Delta x \tag{3.29}$$

The effect of this psuedodispersion is to smooth the profile for positive values of E_n, and to produce instabilities in the results for negative values. This same problem can be shown diagramatically (Figure 3.9) for the case [Pedersen and Prahm, 1973],

$$u = \frac{1}{2}\frac{\Delta x}{\Delta t} \tag{3.30}$$

which gives a positive numerical dispersion coefficient.

From Equation (3.28), it can be seen that one way of reducing the error is to decrease the spatial increment. However, this tends to increase computer costs to astronomical levels as the time interval, Δt, usually has to be reduced also to meet stability criteria. A second method would be to choose Δx and Δt such that

$$u = \frac{\Delta x}{\Delta t} \tag{3.31}$$

and this has been done successfully for some steady, uniform flow problems. In the tidal canal problem, however, the flow is unsteady and nonuniform, and the condition of Equation (3.31) cannot be met everywhere. In fact unless this condition is met at the tidal entrance, E_n will be negative somewhere in the canal network in this example.

The above analysis was based on the backward difference scheme, but serves to illustrate the problem. Similar numerical dispersion expressions can be developed for other schemes for idealised flow conditions. The aim of this study then, is to develop a scheme in which the inherent numerical dispersion present in the numerical solution is much smaller than the natural dispersion being modeled, and also one which does not require excessive computer time because of the smallness of the spatial and time increments, Δx and Δt respectively.

A number of schemes were developed by this author [Walton, 1976a], forward, backward and central explicit, and backward and central implicit, as well as by Langley [1976] who used an explicit central-difference scheme to model flow in a canal with a boat basin. For the first test canal network (Figure 3.3), using the standard data set listed in Table 3.3, it was found in every case that the only way to improve the accuracy of the schemes was to reduce Δx, and therefore Δt (Figure 3.10). For the 1,000 ft canal (Table 3.3), it was necessary to make Δx = 25 ft and Δt = 100 secs.

In conclusion it was decided that this type of restriction on the size of the spatial and time increments would serve to produce an uneconomic model for much larger canal networks and for a three-dimensional model. A similar finite-element model developed by Leimkuhler [1975] was also tested, and was found to suffer from the same drawbacks as the finite-difference models. This latter method was therefore also discounted.

3.3 Method-of-Characteristics Approach

As was seen in Section 3.2, the numerical dispersion inherent in the more common low order finite-difference and finite-elements schemes is sufficiently large to swamp the natural dispersion being modeled, for economic choices of the spatial and time increments. The problem is then to model the convective part, Equation (3.27), of the one-dimension mass-transport equation economically and accurately, and minimize the numerical dispersion produced.

Equation (3.27) is a first-order hyperbolic equation. A traditional way to solve this equation is the Method-of-Characteristics. If the one-dimension mass-transport equation in its nonconservative form, Equation (3.18), is considered, the left hand side of the equation can be exoressed as a material derivative,

$$\frac{Dc}{Dt} = \frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} = \frac{\partial}{\partial x}\left(E_\ell \frac{\partial c}{\partial x}\right) + q_I\ (c_I - c)/A \tag{3.32}$$

A two step solution scheme can then be developed by firstly convecting the concentration field along the characteristic lines of the velocity forming new nodal concentrations $\tilde{c}_i^{\,n+1}$, where i represents a spatial node and n+1 is the new time level. Equation (3.32) is now a second-order parabolic equation with no dominating first-order hyperbolic term, and can readily be approximated by a central-difference scheme, in which the time derivatives are approximated along the characteristic line of the velocity, and the spatial derivatives are estimated by averaging between the two time levels,

$$\begin{aligned} c_i^{n+1} = \tilde{c}_i^{n+1} + \frac{\Delta t}{2\Delta x^2} [(E_{i+1} + E_i)(c_{av_{i+1}}^{\ n+1} - c_{av_i}^{n+1}) \\ - (E_i + E_{i-1})(c_{av_i}^{\ n+1} - c_{av_{i-1}}^{\ n+1})] \\ + \Delta t q_I\ (c_I - c_{av_i}^{\ n+1})/A^{n+1} \end{aligned} \tag{3.33}$$

where

$$c_{av_i}^{n+1} = \frac{1}{2}(\tilde{c}_i^{n+1} + c_{i+\Delta}^{n}) \tag{3.34}$$

in which

Δ = dimensionless distance traveled by characteristic velocity to reach node i.

The convective step can be performed in two ways as illustrated in Figure 3.11. The first method is to convect the concentrations at the existing nodes to form a new nodal grid at the next time level.

This is called the movable grid formulation. The second method is to interpolate between nodal values using the velocity characteristic lines to estimate where new nodal concentrations originated. This is called the fixed grid formulation.

The two variations were run with the standard data set of Table 3.3 for a variety of spatial and time increments. The results shown in Figures 3.12 and 3.13, and others, are analysed in Section 3.7.

3.4 Hybrid Computer Approach

If the convective Equation (3.27) is again considered, a function F(x,t) is defined as follows,

$$F(x,t) = \frac{\partial c}{\partial t} + u\frac{\partial c}{\partial x} = 0 \qquad (3.35)$$

for a uniform velocity field, u, and $\tilde{F}(x,t)$ is defined to be the finite-difference approximation to F(x,t), then

$$F(x,t) = \tilde{F}(x,t) + \Delta\tilde{F}(x,t) \qquad (3.36)$$

where

$\Delta\tilde{F}(x,t)$ = truncation terms of finite-difference series expansion.

For a central difference scheme of the spatial derivative, and a forward difference scheme for the time derivative, using Taylor series expansions up to the second order,

$$\Delta\tilde{F}(x,t) = -\frac{\Delta t}{2}\frac{\partial^2 c}{\partial t^2} \qquad (3.37)$$

as the central difference scheme is second order accurate. Differentiating Equation (3.35) with respect to t and noting that the differentiation is commutative,

$$\frac{\partial^2 c}{\partial t^2} = - u\frac{\partial}{\partial x}\left(\frac{\partial c}{\partial t}\right) \tag{3.38}$$

Using Equation (3.35) again,

$$\frac{\partial^2 c}{\partial t^2} = u^2 \frac{\partial^2 c}{\partial x^2} \tag{3.39}$$

and substituting into Equation (3.37)

$$\Delta\tilde{F}(x,t) = - \frac{\Delta t}{2} u^2 \frac{\partial^2 c}{\partial t^2} \tag{3.40}$$

Thus, for a central difference scheme, an expression has been derived for the numerical dispersion coefficient,

$$E_n = \frac{-\Delta t}{2} u^2 \tag{3.41}$$

This form of the numerical dispersion coefficients illustrates an intriguing possibility. If the time increment could be decreased to zero then the numerical dispersion would vanish irrespective of the spatial increment, Δx. Although Equation (3.41) does not accurately represent the numerical error present in modeling a nonuniform velocity field, it does at least suggest its basic form.

One way to decrease the time interval to zero, or make time continuous, is to use an analog or hybrid computer. This consists of a digital computer controlling the potentiometer settings of an analog computer. A project was initiated with the Hybrid Computations Laboratory at Martin-Marietta Aerospace in Orlando, Florida [Morris, Walton and Christensen, 1977; Sorondo and Baldwin, 1977], to study the feasibility of applying analog modeling techniques to the transport in a one-dimensional canal network. A finite-difference model was also designed for the same system to verify the model.

An analog model has several advantages over the digital models which at that time were being considered for canal networks. Firstly, analog solutions are obtained very quickly, and many hundreds of tidal cycles could be simulated in less than a minute. On a digital computer this would take several minutes of central processing unit (CPU) time, plus a much longer time to print out the results. Secondly, because of the hybrid capability, many test runs can be run in a very short time, allowing a much more complete look at the effects of varying each parameter alone.

The objectives of the hybrid model study were:

a) To determine whether the hybrid approach and the Martin-Marietta facilities were capable of implementing a two-dimensional simulation of pollution transport in a tidal canal network.

b) To evaluate the hybrid model by comparison with digital models being developed by the Hydraulic Laboratory.

c) To determine whether the nonconservative or the conservative formulation of the mass-transport equation was preferred.

d) To determine the most consistent finite-differencing techniques.

e) To determine which of the boundary conditions provides the most realistic results consistent with other models using difference techniques.

f) To assess the relative influence or significance of variations in the independent variables of the analog model.

The analog model was set up as the first test canal (Figure 3.3) with five segments in the canal and a sixth segment in the receiving water to control the tidal entrance boundary condition (Figure 3.14). The model, which included the capability of switching between four different explicit finite-difference techniques, the forward, backward, central and first upwind differencing schemes (the latter being simply a backward-difference scheme for the ebb tide, and a forward-difference for the flood tide), required a large percentage of an analog patch-panel, sometimes called a *hybrid terminal*. An example of a circuit diagram for one node of the canal is given in Figure 3.15. It was estimated that a total of twenty nodes could be programmed onto one panel. The output was in hard copy form on an eight channel strip chart recorder, the eighth channel giving the velocity at the tidal entrance (Figure 3.16). The results for the first test canal case for each of the four different finite-difference techniques are given in Figure 3.17.

After the work was completed a more detailed report was written [Morris, Walton, and Christensen, 1977] and submitted to the Office of Sea Grant. The value of this modeling technique and its possible extension to three-dimensional modeling will be covered in a comparative discussion of all the techniques investigated in Section 3.7. The conclusions are given below.

1. The central difference form of the nonconservative equation appears to give the most consistent results, both in the analog and the digital models. The capability of obtaining a set of concentration plots over a thousand tidal cycles in a matter of seconds (after the model has been initialized) is very convenient.

2. The conservative form of the equation has potential, but its results were obscurred by an unrealistic boundary condition and some instabilities. Further testing of this form would be required before a choice between them can be made.

3. The boundary condition at the open-end of the canal is satisfactory.

4. The zero-flux ($\partial c/\partial x = 0$) boundary condition at the closed-end of the canal appears to distort the results. However, the linear boundary condition gives results which are in very good agreement with other models developed.

5. The mass balance boundary condition at the dead-end was not functioning properly, although it has been shown to operate properly in digital models developed by the Hydraulic Laboratory. Further testing of this formulation would be desirable.

6. The five-segment model is somewhat coarse spatially, but better than was initially expected. A ten or twenty-segment model would be useful for verifying the results of the five-segment model, but could not be expected to provide much additional information.

7. Flushing time, the time for the model to reduce concentrations at all sections to "steady-state" values from some initial, nonequilibrium condition, cannot be inferred from the present model results due to inconsistencies in the variability portion of the study. However, these inconsistencies may be due in part to difficulty in accurately reading plotted results.

8. Expansion of the model to about twenty sections is possible using one analog programming board on the hybrid computer. A two-dimensional model incorporating three nodes in the vertical direction would therefore require three analog consoles. Multiple-board simulations are difficult to schedule and expensive to run. Therefore, it is concluded that for canal networks and two-dimensional models the analog/hybrid approach is extremely limited by hardware availability, not generally practical, and quite expensive.

9. It typically would take several days to a week to have a specific set of analog or digital computer runs completed and mailed to the Hydraulic Laboratory, although it is possible to greatly reduce this time if expense is secondary.

10. Numerous refinements, as outlined in the initial proposal were not tested as they required more hardware and would have obscured the basic evaluation. These refinements included variable canal geometry and additional water quality parameters. Incorporation of these additional variables, which would ultimately be required for a complete canal design capability, would be prohibitive from an analog hardware standpoint.

11. The basic canal *network* formulation was not tested due to lack of a good closed-end boundary condition and lack of time. A single canal junction in one-dimension could be implemented without much difficulty on the analog computer, but extension to a full canal network is definitely limited by analog hardware availability.

3.5 Second Upwind Differencing Method

The "Second Upwind Differencing" method, as termed by Roache [1972] and used by Bella and Dobbins [1968], or "the Donor Cell" method as it was called by Gentry, Martin, and Daily [1966], is a finite-difference technique used by Lee [1977] to solve the one-dimensional mass-transport Equation (3.17). Written in its conservative form in terms of the discharge, Q, rather than the cross-sectional mean velocity, u,

$$\frac{\partial}{\partial t}(Ac) + \frac{\partial}{\partial x}(Qc) = \frac{\partial}{\partial x}\left(AE_{\ell}\frac{\partial c}{\partial x}\right) + q_I c_I \tag{3.42}$$

Using the schematic canal of Figure 3.18 as a guide, the method uses a backward finite-difference expression for the convective term $\frac{\partial}{\partial x}(Qc)$ during the flood tide, and a forward finite-difference term during the ebb tide.

The Second Upwind Differencing method has several advantages over other finite-differencing techniques. Firstly, it is conservative, that is the net accumulation of mass in a region equals the net flux across the boundaries of the region during the convective process.

Secondly, it is transportive during the convective phase, that is "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.

The method suffered originally from the major disadvantage that numerical dispersion was large, as mentioned by Roberts and Weiss [1966]. This problem was examined for the First Upwind Differencing method by Noh and Protter [1963] who derived an expression for the numerical dispersion present in the solution. The theory was extended here to the Second Upwind Differencing method by Lee [1977]. Bella and Dobbins [1968] had suggested that the numerical dispersion might be subtracted from the natural dispersion to give a composite dispersion term that accurately modeled prototype conditions. However, in their studies on rivers and estuaries, the natural dispersion was much larger than the numerical dispersion, giving positive values for the composite dispersion. The problem is that this is not the case in Floridian canals, in which the resulting composite dispersion is negative for reasonable choices of Δx, giving rise to unstable solutions. As mentioned previously, a negative dispersion term has the effect of augmenting instabilities present in the numerical scheme rather than smoothing them as positive dispersion does.

Recently, Boris, Book and Hain [1973, 1975, 1976] developed a correction technique termed "flux-corrected transport". This technique extended the idea of Bella and Dobbins in subtracting the numerical dispersion to include the extra condition that "the anti-diffusion stage should generate no new maxima or minima in the solution, nor should it acccentuate already existing extrema". To accomplish this, the antidispersion flux terms are corrected or limited so that no antidispersion transfer of mass can push the concentration at any grid point beyond the concentration value at neighboring points.

Using the schematic canal of Figure (3.18) as a guide, the Second Upwind Differencing model with flux-corrected transport may be described as operating as follows during one time step:

1. addition of pollutant through lateral inflow,
2. net tidal convection of pollutant into the segment from adjacent segment with values of concentration, c, defined at the center of the segment, and values of the discharge, Q, defined at boundaries of the segment,
3. limited antidispersion to correct for the numerical dispersion errors introduced in the convective step but controlled by the flux-corrected transport criterion,
4. net transport of pollutant into the segment by natural longitudinal dispersion.

This method is essentially second-order accurate, as it has been corrected for the second-order error, the numerical dispersion. Even with the correction term, the method remains both conservative in all steps and transportive in the convection step, properties which cannot be claimed in all cases by the other finite-difference and finite-element models so far considered. These properties alone do not ensure that the desired solution will be obtained. The stability and convergence of the solution must also be considered.

Although there is no universally accepted definition of stability, this term is generally used to refer to the damping with time of small perturbations (or errors) introduced into the finite-difference solution. It can readily be seen from Figure 3.19 that the solution shows no visible instabilities to be present. One is also concerned about convergence, or whether the solution approaches the solution of the partial differential Equation (3.17) and Δx and Δt approach zero.

For equations such as the one-dimensional mass-transport Equation (3.17), with variable coefficients, necessary and sufficient stability and convergence conditions are not available. However, conditions such as

$$|u_{max}| < \frac{\Delta x}{\Delta t} \tag{3.43}$$

and

$$E_{\ell max} < \frac{\Delta x^2}{2\Delta t} \tag{3.44}$$

make sense physically and provide guidelines for choosing Δx and Δt.

In the final analysis, comparisons of the numerical solutions of the difference models must be relied on in attempting to assess the accuracy and, considering computing costs, the economics of any one model. This comparison is made in Section 3.7.

3.6 Method of Second Moments

The Method of Second Moments is essentially an upwind finite-difference scheme. However, instead of considering a concentration respesented at a single point, or node, the concentration is considered to be a block with the height being the magnitude of the concentration and the width being the width of the cell, Δx, the spatial increment. Thus, like the nodal finite-difference representation, this method suffers equally from numerical dispersion.

The advantage of this way of representing the concentration is that it conserves the mass, or the zeroth moment of the mass. One method that has been used in meteorology to reduce the numerical dispersion present in finite-difference schemes, is to also conserve higher order moments of the distribution [Egan and Mahoney, 1972; Pedersen and Prahm, 1973]. Any number of moments could be chosen to represent the distribution, but commonly the first and second moments are used. These physically represent the center of the mass, and the width of mass with respect to its center. Higher order moments, such as third order moments which represent the skew of the distribution, do not have the same intuitive feel, but could be used if the lower order moments fail to represent the distribution accurately.

Consider the concentration distribution, $c(\xi)$, shown in Figure 2.20 in a cell of unit width, represented by a rectangle having the same mass, c_m, the same center of mass, F_m, and the same second moment squared, R_m^2 [Pedersen and Prahm, 1973], then,

$$c_m = \int_{-0.5}^{0.5} c(\xi)d\xi \tag{2.45}$$

where

$$\xi = \frac{x - (n-1)\Delta x}{\Delta x} \tag{2.46}$$

and

$$F_m = \int_{-0.5}^{0.5} c(\xi)\xi d\xi / c_m \tag{2.47}$$

$$R_m^2 = 12 \int_{-0.5}^{0.5} c(\xi)(\xi - Fm)^2 \, d\xi / c_m \qquad (3.48)$$

Figure 3.9 illustrates what happens if only the convection of a square wave as defined by Equations (3.27) and (3.30) is considered. This form conserves the zeroth moment of the distribution, or its mass. If the first moment of the distribution is also conserved (Figure 3.21), it can be seen that the analytic distribution is much more closely approximated. Inclusion of the second moment for this particular problem reproduces the square wave exactly, and is another reason for considering only moments up to the second order.

The problem defined in Equations (3.27) and (3.30) is a uniform flow case. For an oscillatory flow in canals the method has to be ammended. A closed form solution for the velocity at any point in the canal was developed in Section 3.1.3 (Equation (3.9)). Assuming that the lateral inflow entering each cell has a uniform longitudinal variation in that cell, then the velocity variation over the length of the cell is linear in x and represents the distortion the cell undergoes as the tidal elevation changes from the old cross-section area, A_p, to the new area, A. As each reach of the canal network is prismatic, the ratio A_p/A represents the amount of expansion or contraction of each cell in the reach. Thus, the first step in the solution procedure is to distort the cell,

$$c_m = c_{mp} A_p / A \qquad (3.49)$$

$$F_m = F_{mp} \qquad (3.50)$$

$$R_m = R_{mp} A_p / A \qquad (3.51)$$

where

p = previous time level.

The second step is to move this new distribution in the direction of the flow a distance equal to the distance moved by the center of mass in the time interval, Δt

$$x = \Delta x(1/2 + F_m) - V/A \tag{3.52}$$

where

V = incremental volume of tidal prism upstream of cell (V is defined as positive for a flood tide), (L^3).

The new distribution and the lateral inflow are then allocated to the various cells into which they fall, where for each cell the square of the second moment, $R^2_{c_i}$ of each composite distribution is calculated about the center, c, of cell i,

$$R_{c_i} = 12 \sum_{j=1}^{N_i} \int_{-0.5}^{0.5} c_j(\xi)\xi^2 \, d\xi/c_{m_i} \tag{3.53}$$

where

N_i = number of distributions that lie inside cell i.

When all the cells of each reach have been readjusted, the square of the second moment of the resulting distribution about the new center of mass, F_{m_i}, is found using the parallel axis theorem for each cell,

$$R^2_{m_i} = R^2_{c_i} - 12 F^2_{m_i} \tag{3.54}$$

Finally, dispersion is modeled at the new time level using the central finite-difference scheme as before,

$$c_i^{n+1} = \tilde{c}_i^{n+1} + \frac{\Delta t}{2\Delta x^2} [(E_{i+1} + E_i)(\tilde{c}_{i+1}^{n+1} - \tilde{c}_i^{n+1})$$
$$- (E_i + E_{i-1})(\tilde{c}_i^{n+1} - \tilde{c}_{i-1}^{n+1})] \tag{3.55}$$

where

$\tilde{c}_i$ = is the concentation value at node i after the convective step.

As this method is an upwind differencing method, the method also possesses the properties of being conservative and transportive as defined in Section 3.5. Similarly, the same stability and convergence conditions must be expected to apply to this method. The results of varying the spatial and time variables, Δx and Δt, for the first test canal (Figure 3.3) using the standard data set of Table 3.3 are shown in Figure 3.22. A comparative analysis of this and the other methods described in the preceeding sections follows.

3.7 Comparison of Techniques

In Sections 3.2 through 3.6, several numerical techniques to evaluate the one-dimensional mass-transport Equations (3.17) or (3.18) were discussed, and the results shown for the simulation of the first test canal, Figure 3.3 using the standard data set in Table 3.3.

Several methods are immediately discounted. The finite-element and finite-difference methods up to second order were discounted because of the restrictive choice of the spatial and time increments, Δx and Δt, respectively, to achieve comparable results with the other models investigated. An extension to a complex canal network would require an excessive amount of computer time. Higher order methods were not considered in this analysis because they require small Δx and Δt increments so that the information being used to model conditions at one node is not spread out over too great a distance. The movable-grid method-of-characteristics approach was also discounted because it was found that the net convection out of system due to the lateral inflow, also convected the grid out of the system over time.

The other four methods, the hybrid method, the fixed grid method-of-characteristics, the second upwind differencing method and the method of second moments, produced very similar results for the first test canal with the standard data set. In obtaining this result, the hybrid model used seven nodes. At the Martin-Marietta facility, it was estimated that twenty nodes could be used on one patch-panel and that three patch-panels could be linked together in series. However, this would tie up the laboratory and provide only sixty nodes which are clearly not enough for a model of any complexity. Several ideas were considered in which the solution domain could be divided up, using the digital computer to control the operation, and the solution matched at boundary nodes. These ideas were dropped when it became apparent that the complexity of such a set-up would be unrealistic. Also, once a patch-panel is set up, the geometry becomes fairly rigid, and it is not easy to program for varying spatial increments and numbers of nodes per reach. For all these reasons, it was decided that the disadvantages outweighed the primary advantage of speed, and further research was discontinued.

The three remaining models are cheap to run and gave very similar results for the first test canal. Therefore, a second run was designed in which there was initially a square wave of a substance of concentration twentv units located for x in [400, 600] at low tide. The background concentration was five units. After ten tidal cycles, the resulting profiles are shown in Figure 3.23. From this figure, two things can be seen. Firstly, the effect of the limited antidispersion is shown for the two runs of the second upwind differencing scheme. Secondly, the profile for the method-of-charcteristics is severely attenuated and dispersed. The reason for this is that the interpolation procedure for redefining nodal values cannot handle the steep gradients of the leading edge found in the square wave.

This can be shown again if another test is performed with the first test canal in which convection only is modeled. The initial concentrations in the canal are background, and a constant lateral inflow load is applied uniformly along the length of the canal. The resulting profiles after 290 tidal cycles are shown in Figure 3.24. Once again, it is clear that the interpolation procedure cannot handle the sharp gradients of the distribution at the tidal entrance, and thus the method-of-characteristics solution technique was also discounted here. It can also be seen from Figure 3.24 that the method of second moments is the most successful method in conserving mass.

From Figure 3.23, and theoretically also, as explained in Section 3.6, the method of second moments convects the square wave very accurately. Added to this, the second upwind differencing method's limited antidispersion step is somewhat artifical and has a fairly complex form in junctions. To examine the effect of junctions, a second data set, Table 3.4 was developed for the second test canal, Figure 3.4. The results are shown in Figures 3.25 and 3.26. The results show that the junction in the second upwind differencing model is causing a change in the gradients of the profile at that point.

Taking all the above factors into account (Table 3.5), it was decided that the method of second moments, because of its extreme accuracy in modeling the square wave, its superior ability to conserve mass, its intuitive relations to physical properties of the distribution, its ease of computation and its relative economy, was ideally suited for an extension to a three-dimensional mass-transport model of a coastal finger canal network.

3.8 Effect of Varying Model Parameters on One-Dimensional Mass Transport

In the previous sections, a one-dimensional mass-transport model has been presented. After an extensive investigation of some of the numerical solution techniques available, it was decided that the method of second moments gave the most accurate results, was stable, convergent, transportive and was also fairly economical to run.

It has been pointed out in the introductory chapter, that the physical phenomena associated with mass-transport in a coastal finger canal network are fully three-dimensional processes under the influence of wind, salinity gradients, tides, secondary currents, and lateral inflows. Their features will be incorporated into a three-dimensional model and the results generated using the numerical scheme selected in this chapter.

However, a study of the variability of the one-dimensional model's parameters is useful here because the effects will be similar in a three-dimensional model, the only difference being the actions of all the other effects modeled. These may be studied independently using the latter model. The variation of these parameters, then, will give a qualitative guide to the design engineer as to relative effects of design elements. Once a canal network is designed, its quantitative functioning can be evaluated using the three-dimensional model. All the variability runs unless otherwise stated used the first test canal network (Figure 3.3) and varied the standard data set (Table 3.3).

The parameters may be divided into two groups called non-controllable and controllable parameters. The first category includes the tidal amplitude, and the decay coefficient, τ, associated with the flood tide, tidal entrance boundary conditon. From Figure 3.27, it can be seen that the decay coefficient, τ, has a surprisingly small effect on the equilibrium concentration profile in the canal over the range of half a tidal period. This means that the inaccuracies inherent in the selection of such a flood tide boundary condition do not have a significant effect of the mass transport in the canal network. On the other hand, the results from varying the tidal amplitude, a, (Figure 3.28) show a much more dramatic effect as would be expected from a simple volumetric tidal prism analysis. Areas with low tidal ranges, and thus automatically low energies, could expect severe pollution problems at the dead-ends of canal systems, unless either suitable flow-through systems can be designed, or other sources of energy, such as the wind, can be utilized to improve flushing.

The second category, called the controllable parameters, includes the geometric parameters of length, width, mean tidal depth, and inverse side slope, as well as the lateral inflow rate and its concentration, Nikuradse's equivalent sand roughness, and the dimensionless dispersion coefficient. The geometric parameters, with the exception of the mean tidal depth, give results that would appear to be intuitively obvious. The effect of increasing the length of the canal, L, was simply to linearly increase the concentration at the dead-end (Figures 3.29 and 3.30), whereas the effect of increase of the bottom width, b, (Figure 3.31) or the inverse side slope, s, (Figure 3.32) for constant lateral inflow and concentration, was to proportionally decrease the equilibrium profile as a greater volume of water is presented to dilute the inflow. Conversely, the effects of increasing the lateral inflow, q_I, while holding its concentration constant (Figure 3.33) or holding the inflow rate constant and increasing its concentration, c_I, (Figure 3.34), for fixed canal geometry, proportionally increased the resulting equilibrium profile.

The effect of increasing the mean tidal depth, d_o, has a different effect (Figure 3.35). The other geometric parameters merely increased the size of the tidal prism and thus the velocity, for a fixed mean depth. The variation in the mean tidal depth alters the velocity of the flow, however, it also alters the dispersion coefficient, and the two effects partially cancel. From Figure 3.35 it can be seen that for increasing depth, the resulting equilibrium concentration profile decreases slowly. This result however is somewhat misleading because of some of the other physical phenomena not modeled. It is usually concluded for many reasons, such as low flows near the bed if the actually vertical velocity profile is considered, the depth of penetration of light to provide photosynthesis, the effect of rearation over a deep water column, and so on, that deep canals are not desirable as anoxic conditions are known to commonly occur. Thus, this result should not be included in an analysis of proposed canal design except to provide an estimate of the equilibrium profile, but rather the effect of other forcing functions should be studied using the three-dimensional model.

The effects of varying Nikuradse's equivalent sand roughness, k, (Figure 3.36) and the dimensionless dispersion coefficient, K (Figure 3.37), have similar effects, as would be expected through their association in the longitudinal dispersion coefficient. Equilibrium concentration profiles increase with decreasing values of these parameters, indicating the fact that rougher canals and bends provide more shear and turbulent eddies that aid the mixing process.

It should be noted that as the magnitudes of the resulting equilibrium profile increase, the time to achieve equilibrium also increases. This could have a potentially serious effect on canal networks which are subject to high loads from time to time. From a statistical point of view, looking at return period relationships, an efficient canal would be one which flushes out a design percentage of the pollutant before the next load was expected. In an inefficient system, a certain amount of the pollutant above the design amount considered acceptable would be retained when the next load arrived. The concentrations would then build up in the system until an equilibrium condition is reached, possibly much above that considered environmentally acceptable by design standards.

The final test used the second model canal network (Figure 3.4) in which a main 1,000 ft canal had a branch at varying locations along is length. The length of the branch was 500 ft. The resulting concentration profiles at low tide and high tide for location of the branch canal at x = 200, 400, 500, 600 and 800 ft are shown in Figures 3.38 and 3.39 respectively. The results indicate that as the branch canal is placed nearer to the dead-end, the resulting concentration profile in the main canal is reduced as the effective excursion of the tidal prism is increased. However, the converse is true for the branch canal and the problem becomes a trade off dependent on acceptable design criteria.

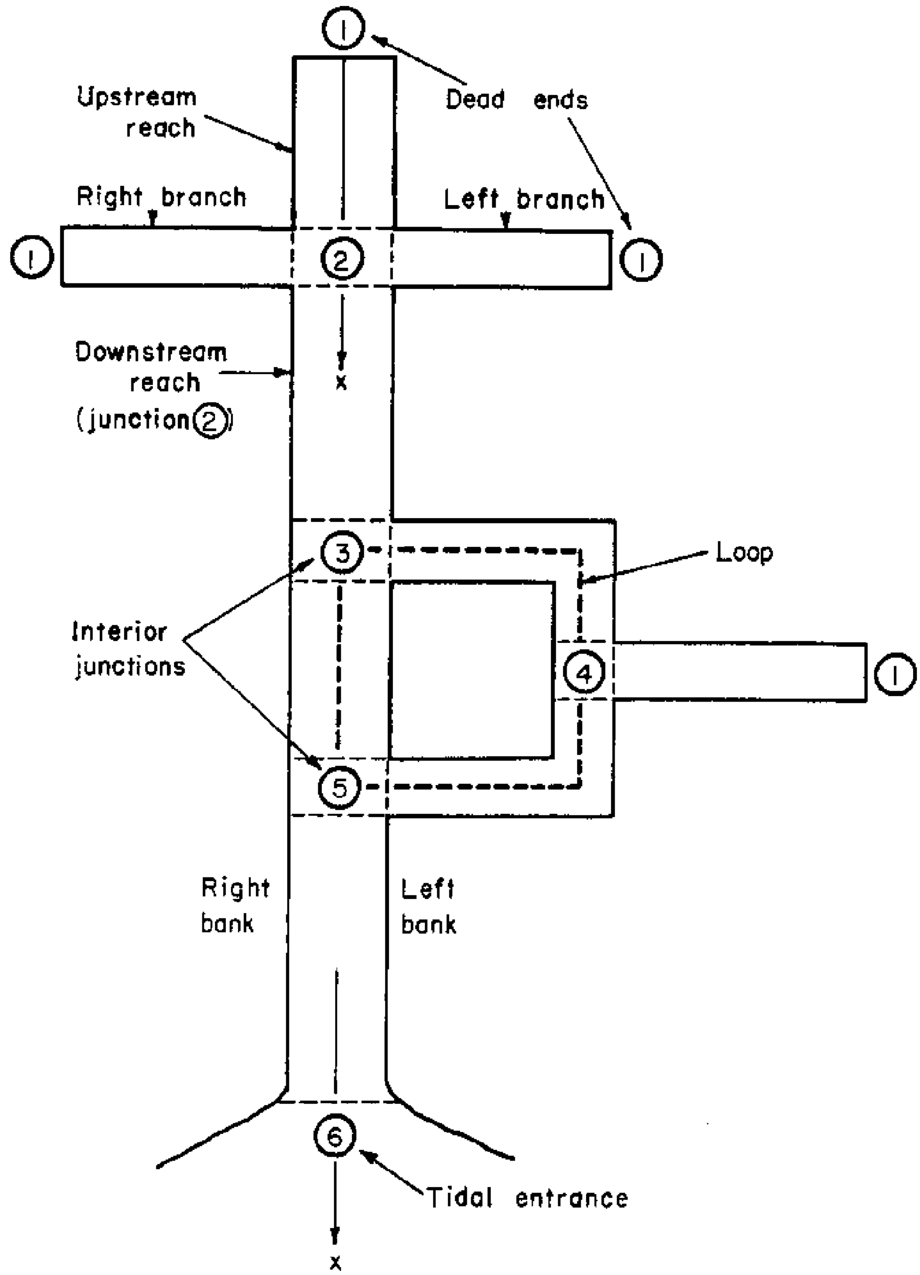


Figure 3.1 - Definition Drawing of Canal Network.

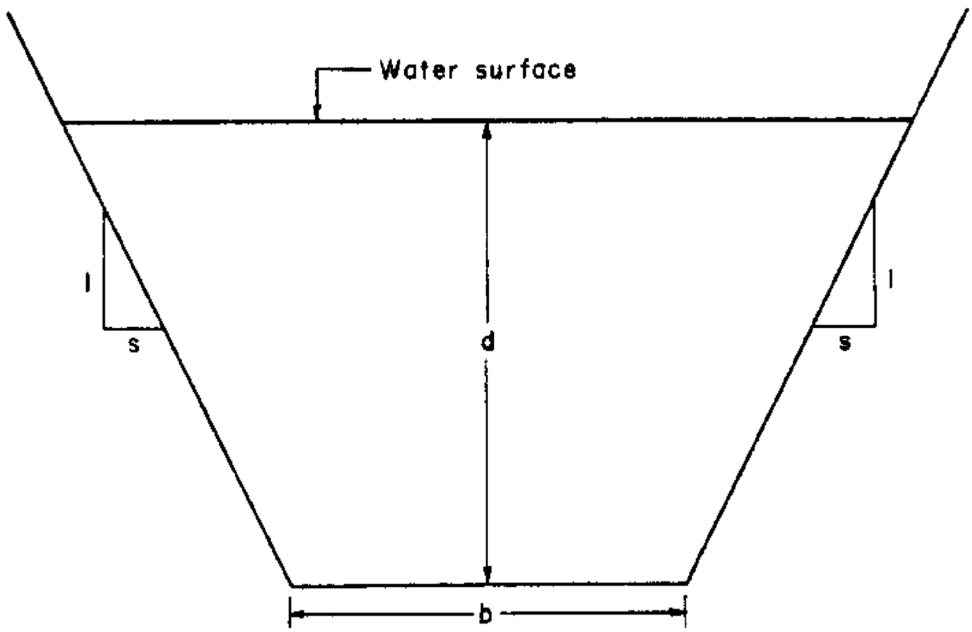


Figure 3.2 - Schematic Drawing of Trapezoidal Cross-Section.

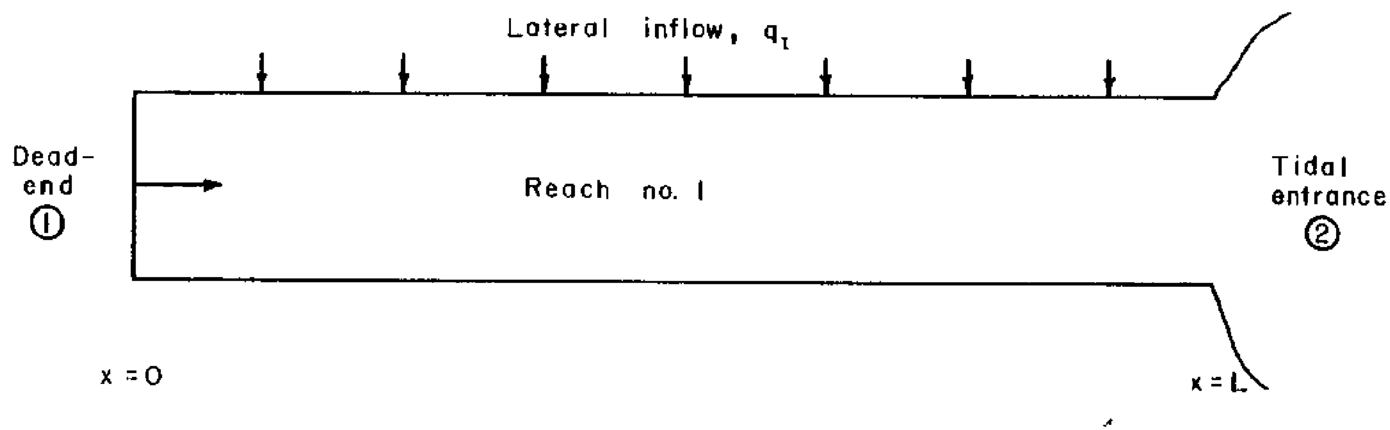


Figure 3.3- First Test Canal Network.

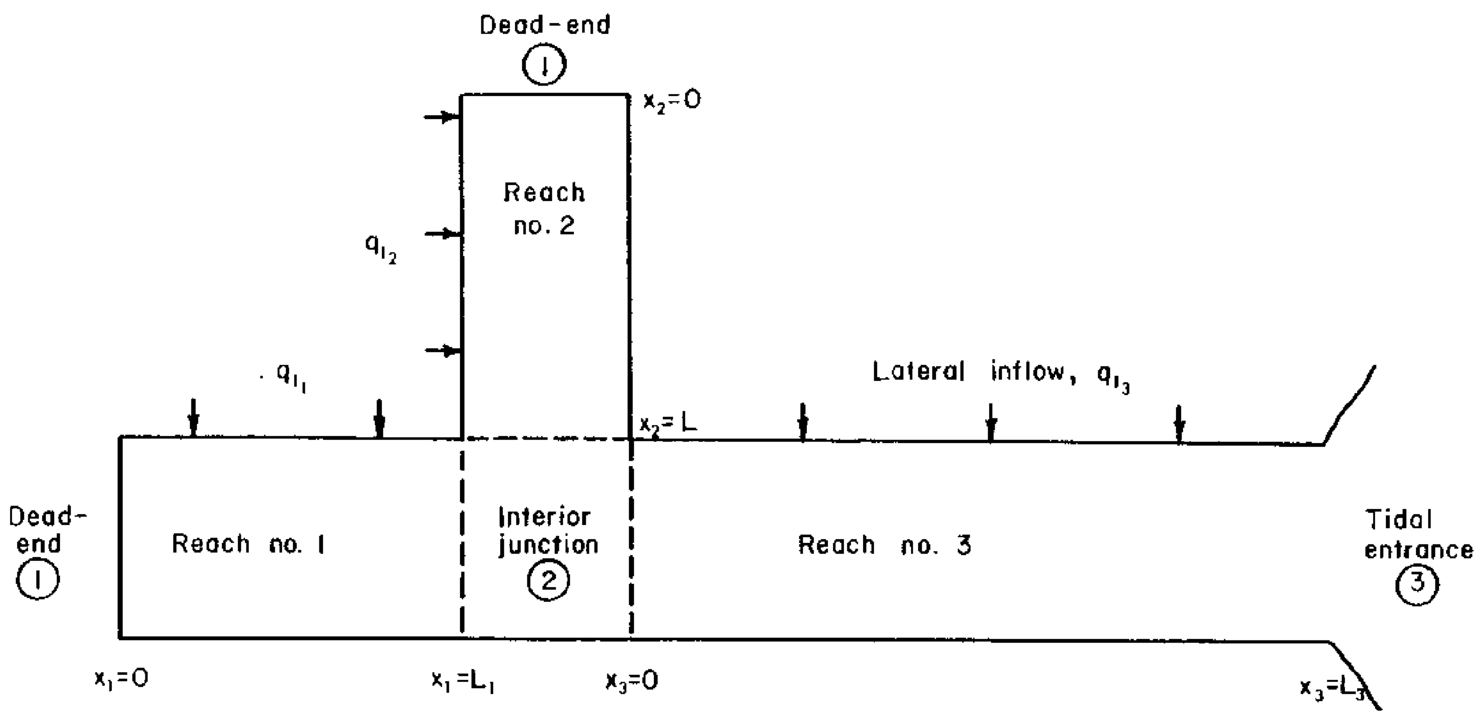


Figure 3.4 - Second Test Canal Network.

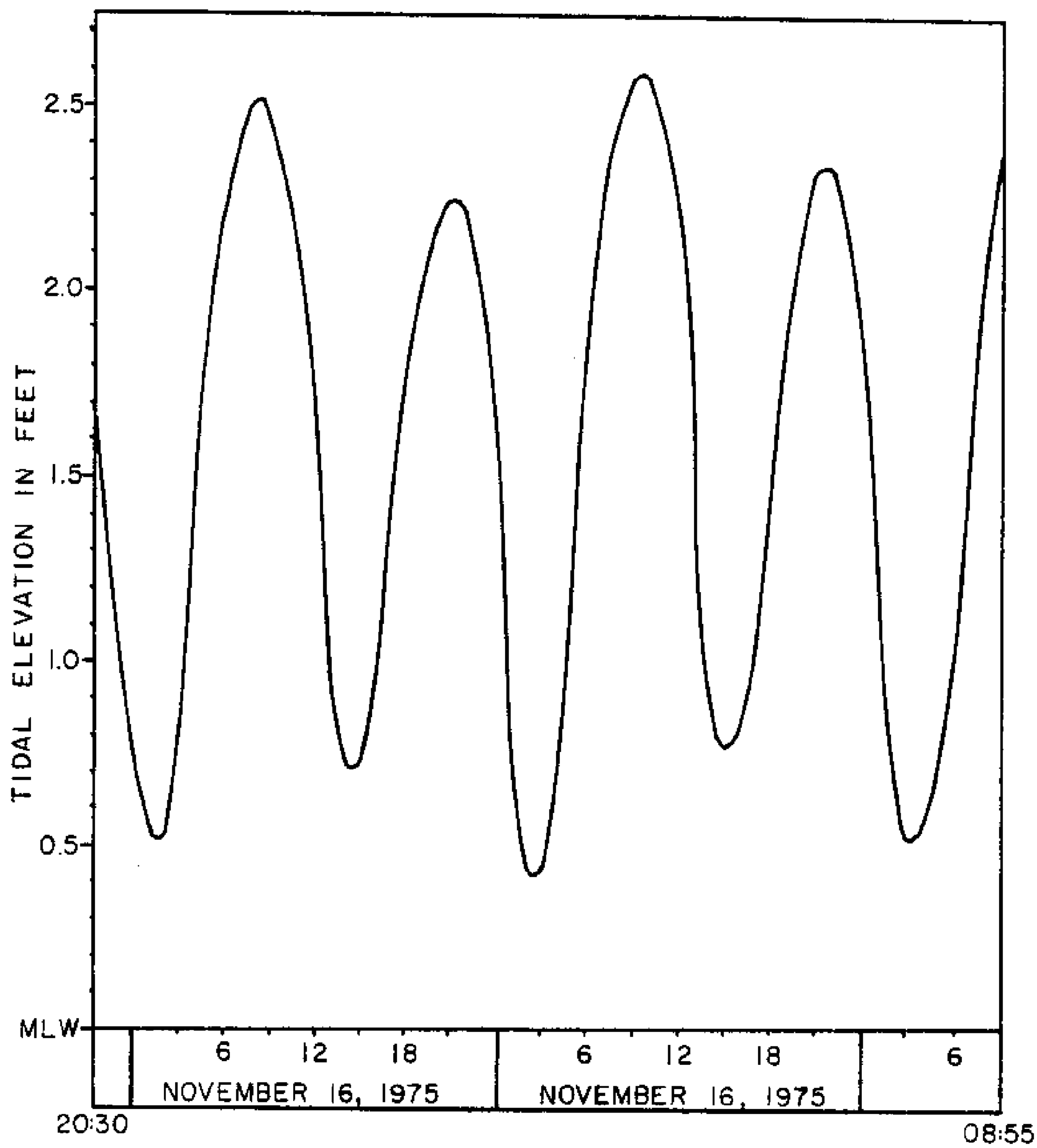


Figure 3.5 - Typical Atlantic Coast Tide Curve.

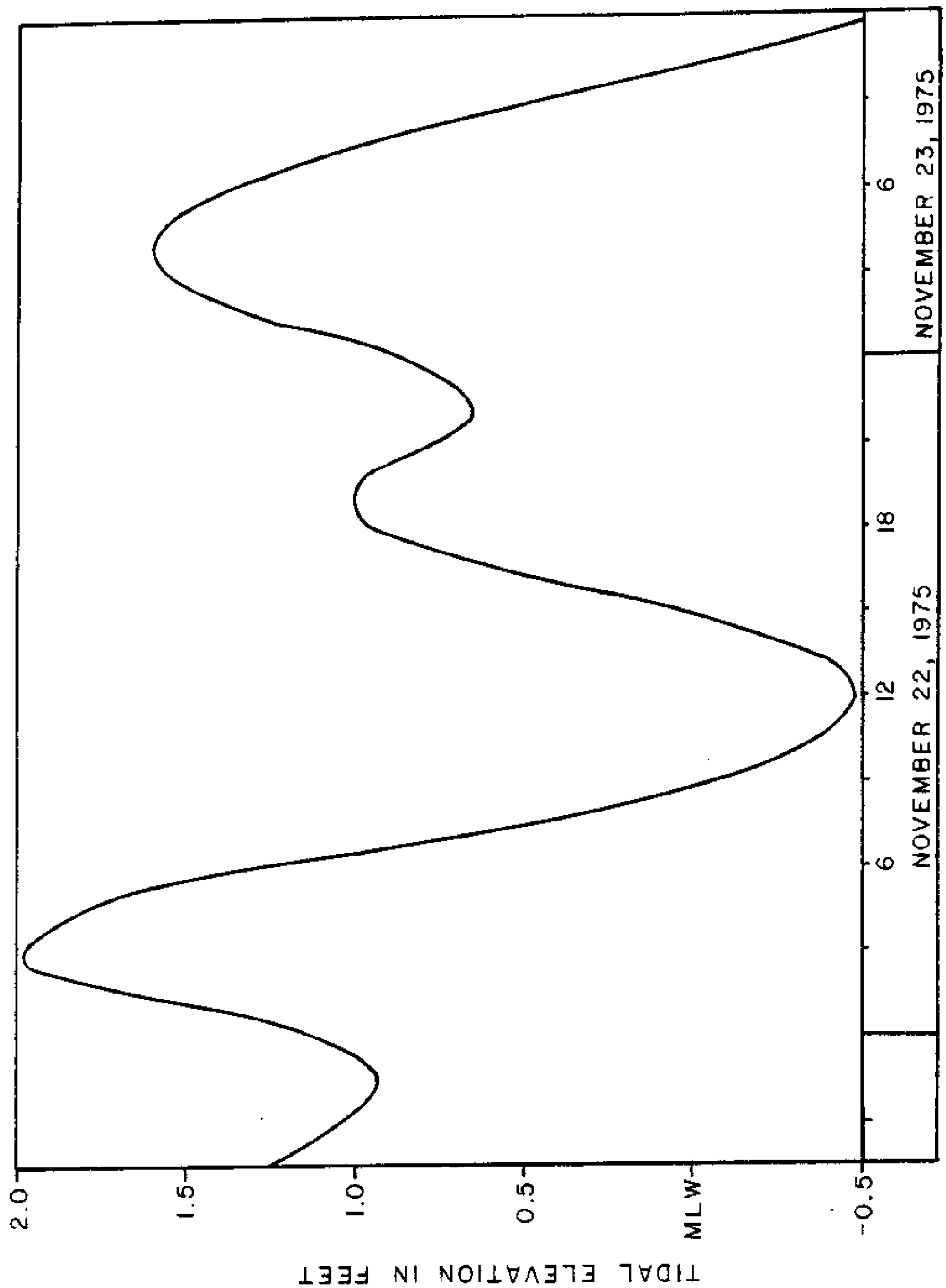


Figure 3.6 - Typical Gulf of Mexico Tide Curve.

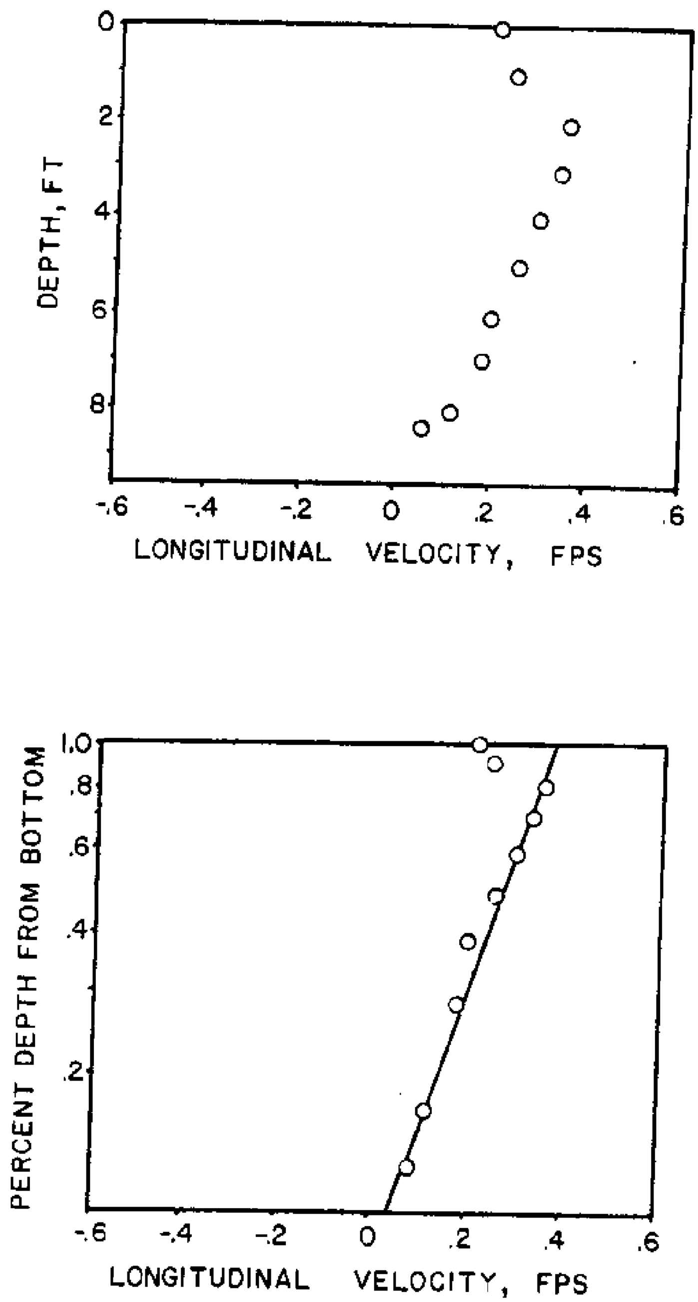


Figure 3.7 - Typical Logarithmic Velocity Profile.

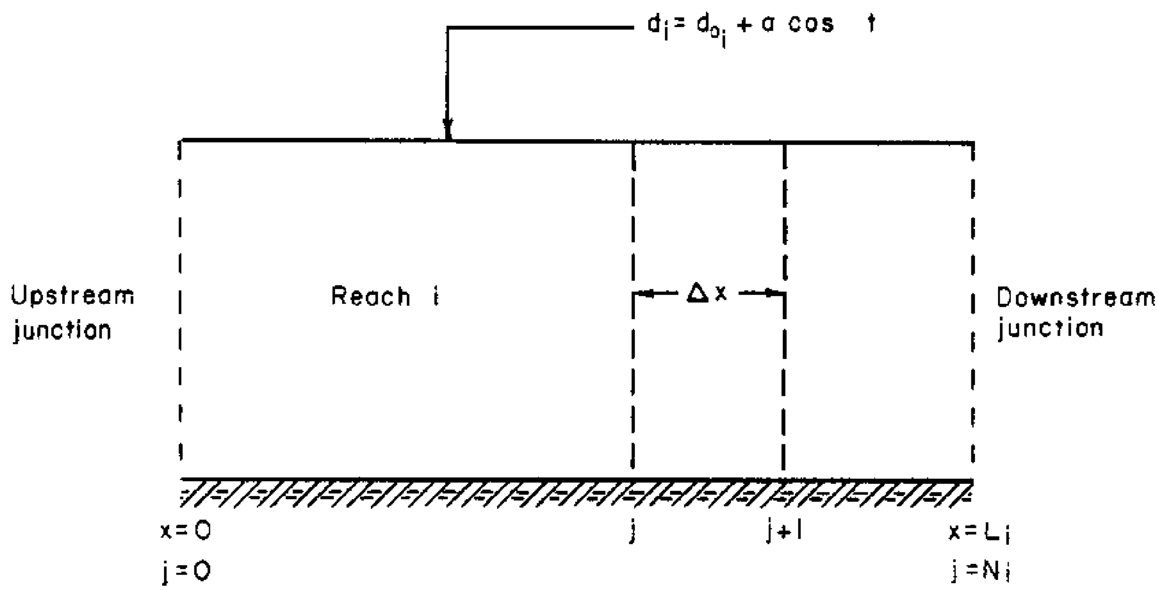


Figure 3.8 - Schematic Drawing of Horizontal Water Surface Assumption.

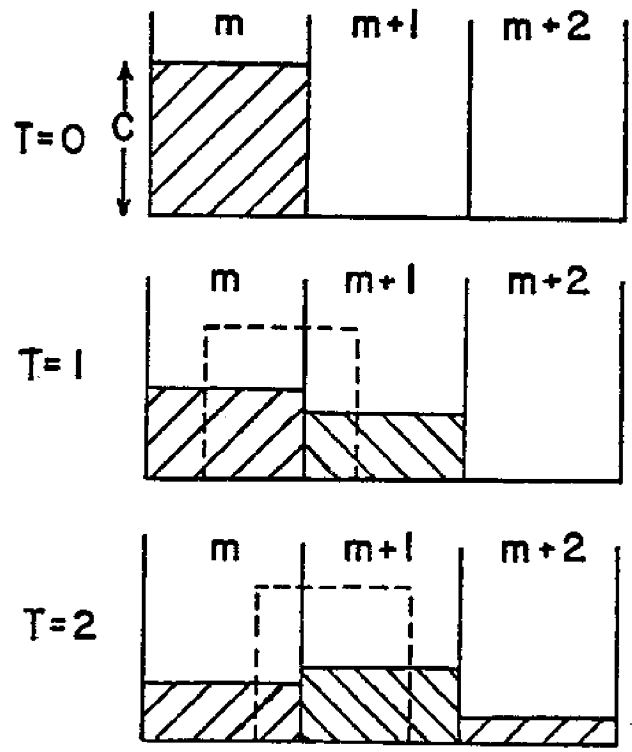


Figure 3.9 - Schematic Representation of Effect of Numerical Dispersion.

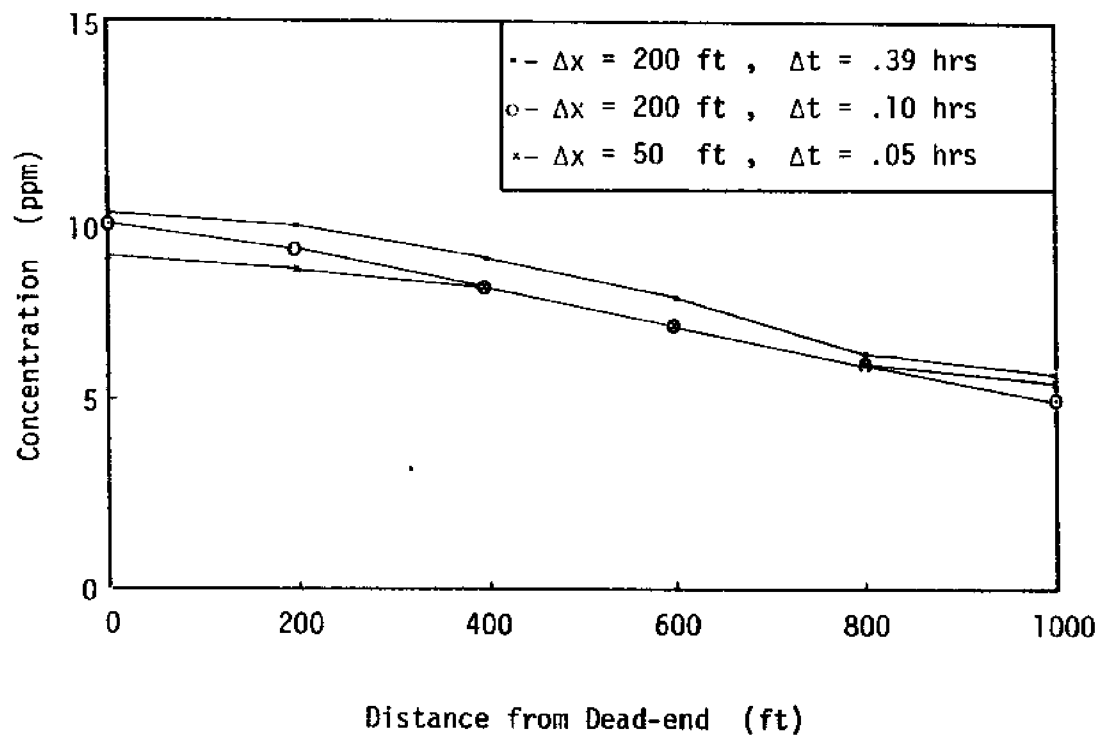


Figure 3.10 - Low Tide Concentration Profiles for Various Δx and Δt - Finite-Difference Method.

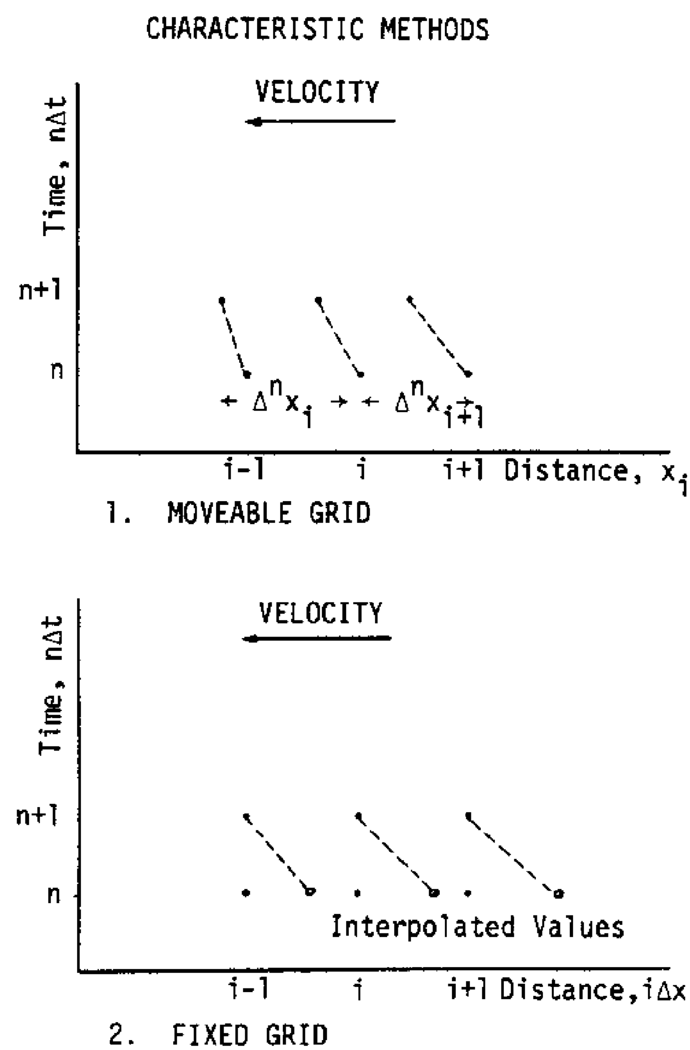


Figure 3.11 - Schematic Representation of Convective Steps of Method-of-Characteristics Methods.

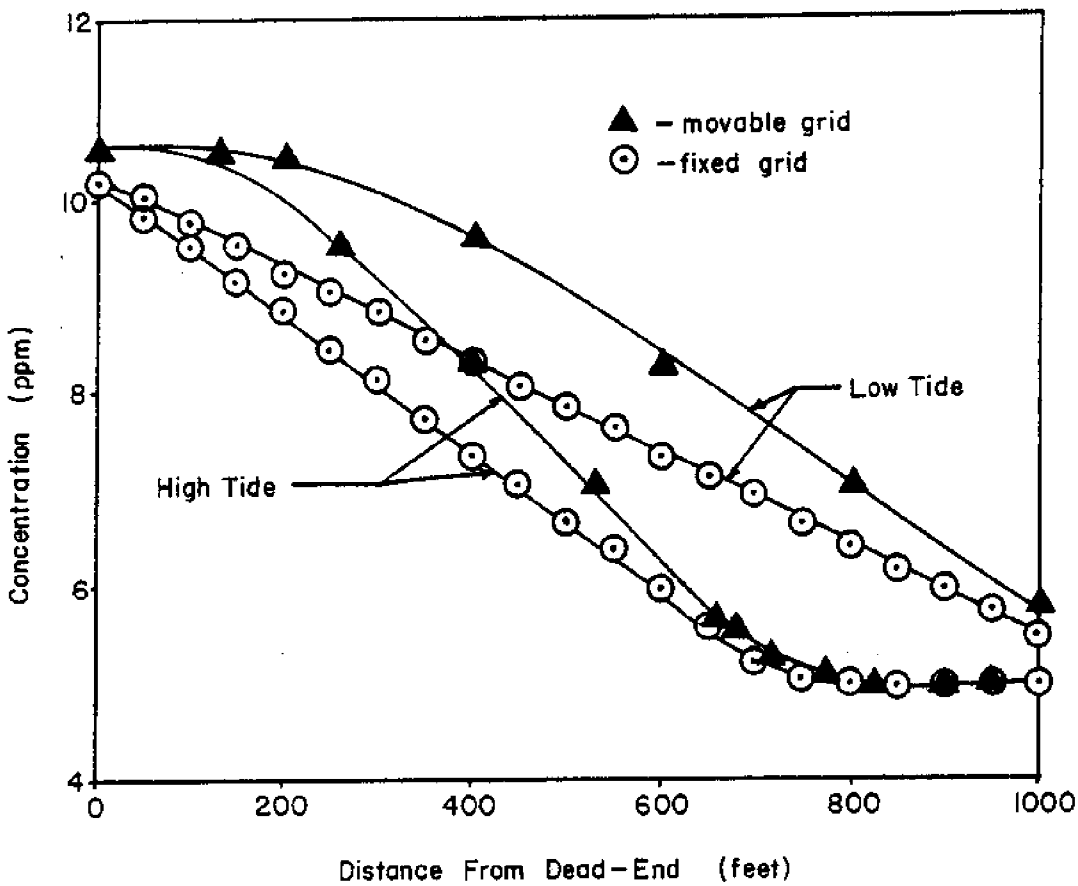


Figure 3.12 - Comparison of Concentration Profiles for Fixed Grids and Movable Grid Method-of-Characteristics.

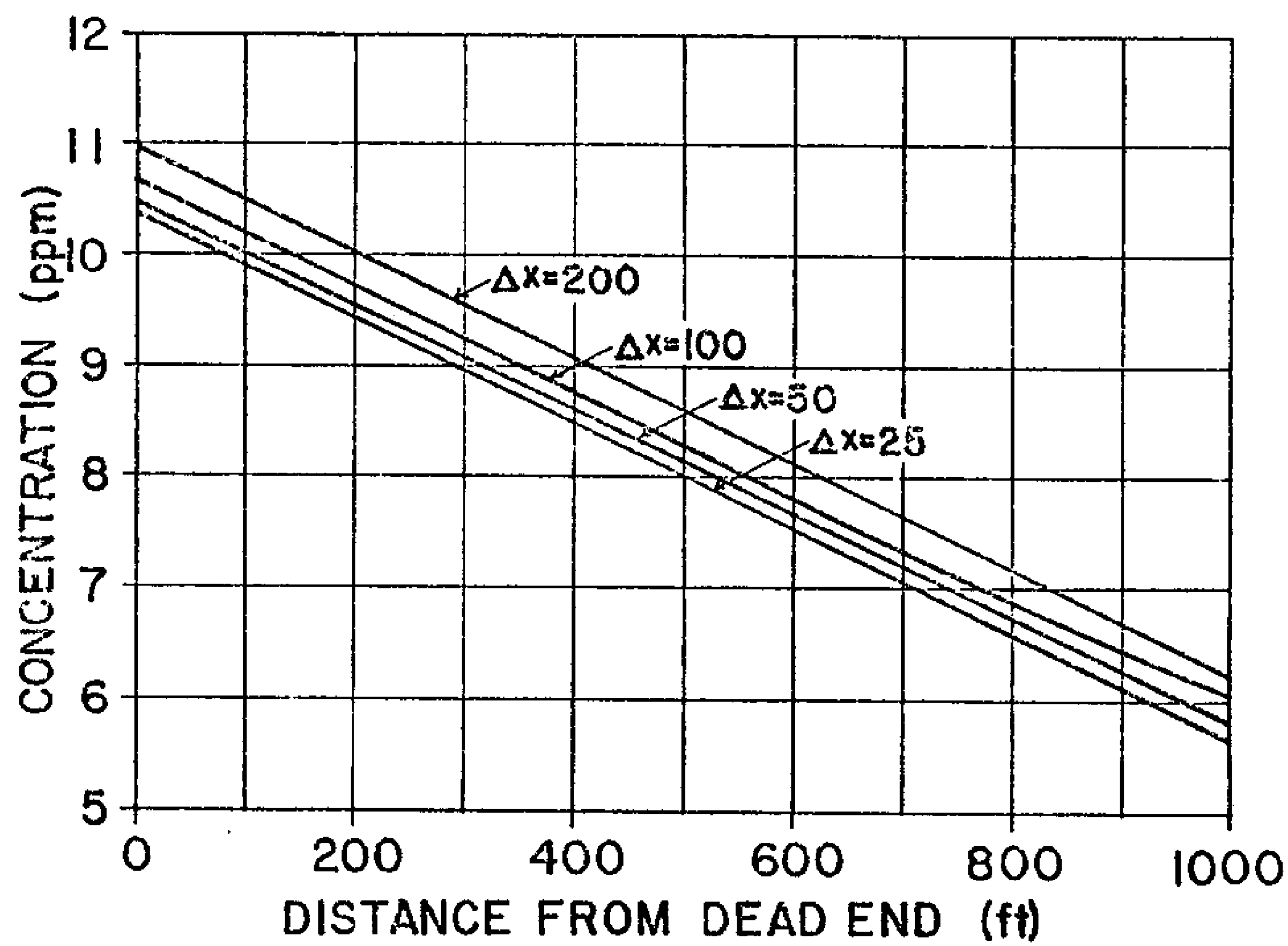


Figure 3.13 - Low Tide Concentration Profiles for Various Δx Fixed Grid Method-of-Characteristics.

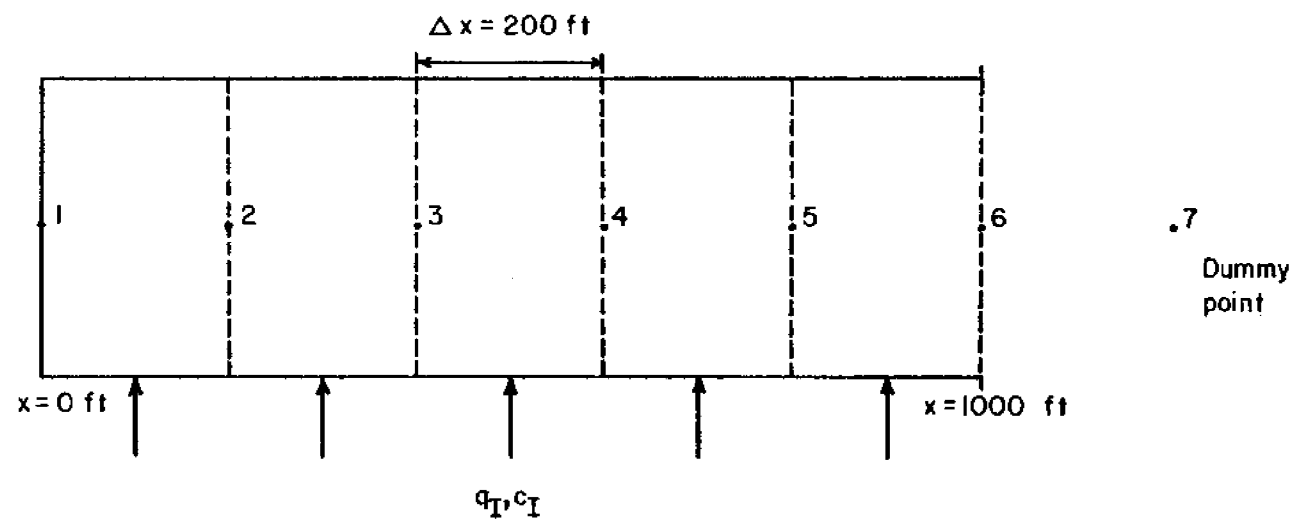


Figure 3.14 - Schematic Drawing of Hybrid Method Test Canal.

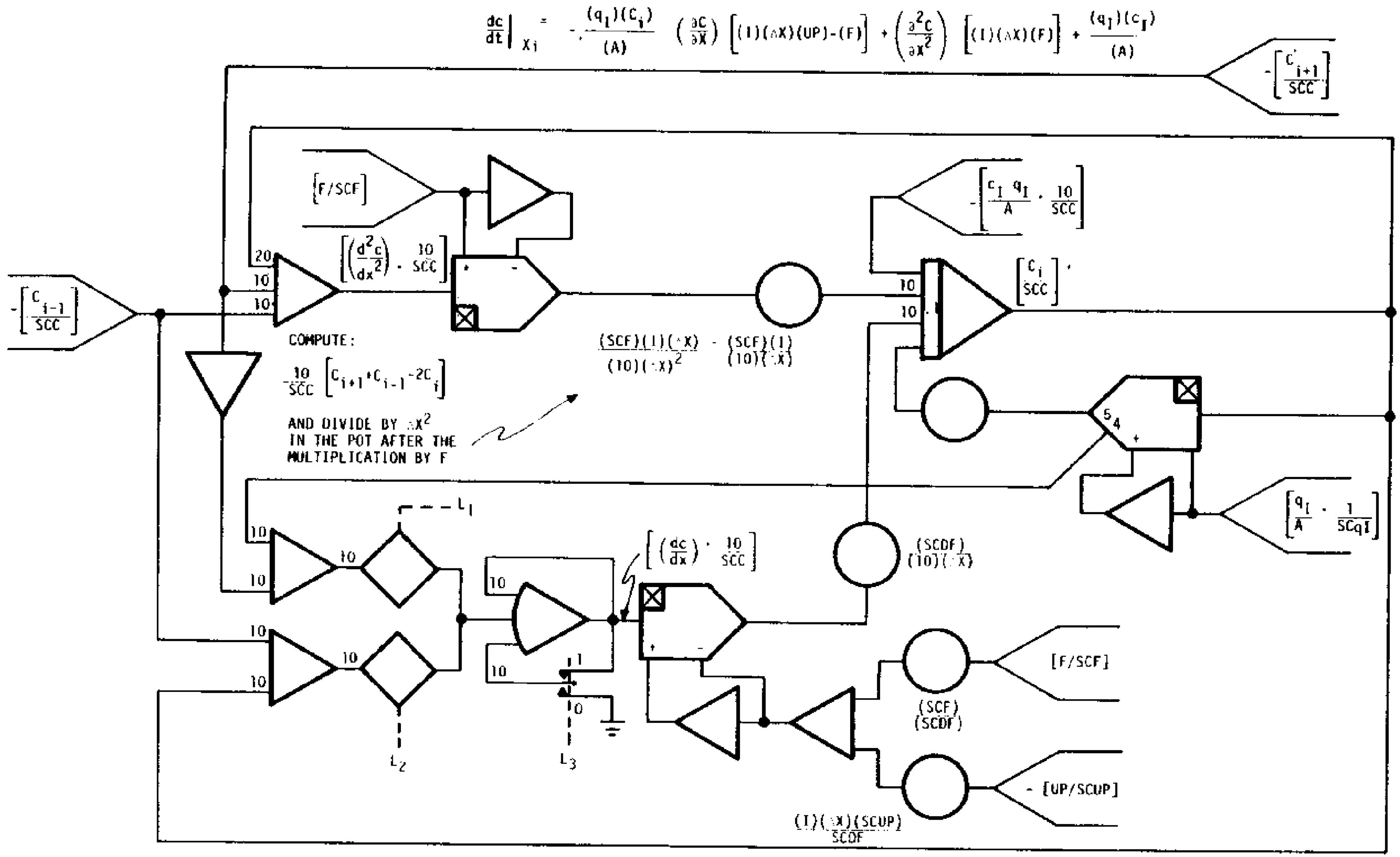


Figure 3.15 - Generalized Analog Diagram of a Single Node of Hybrid Method.

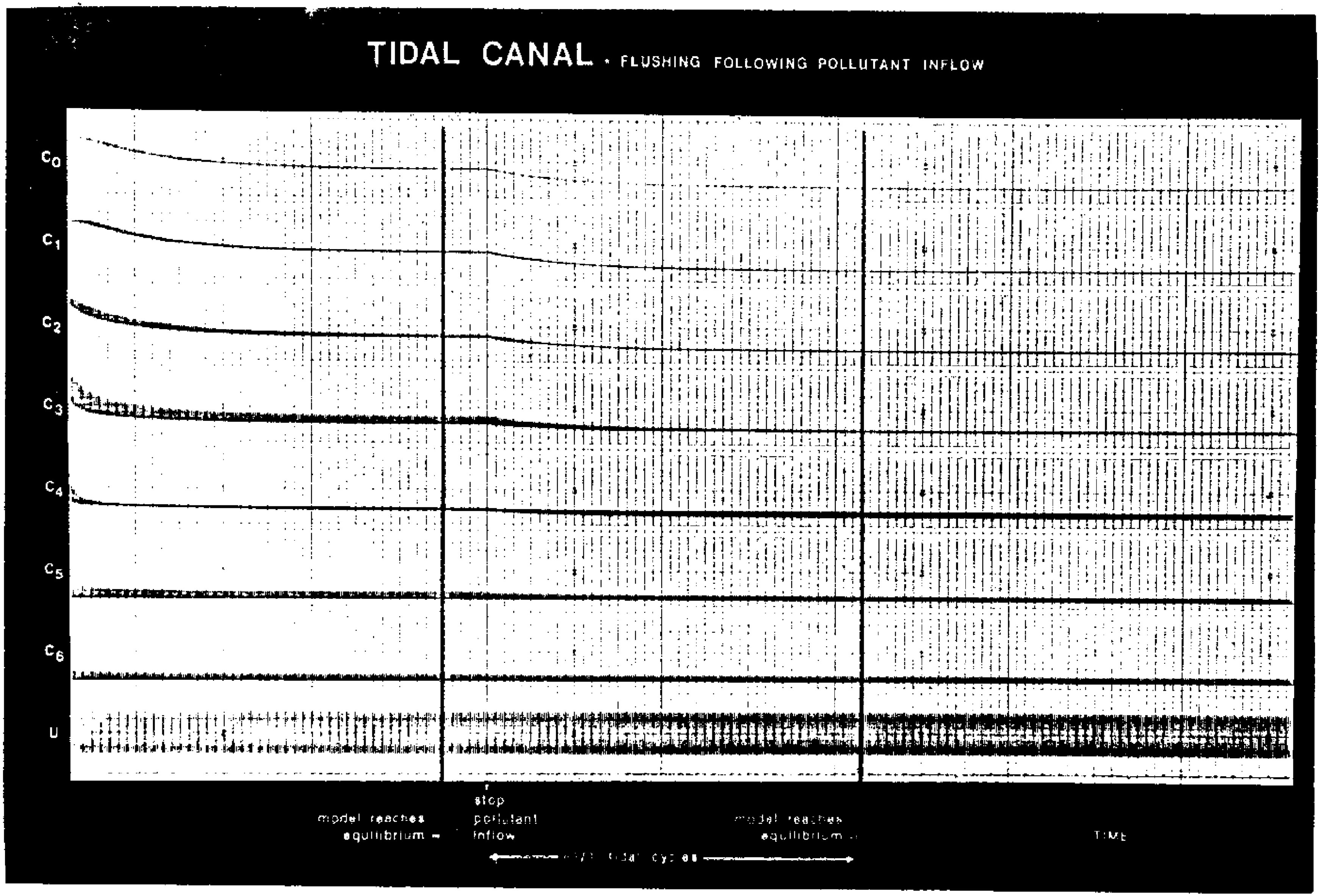


Figure 3.16 - Typical Analog Output

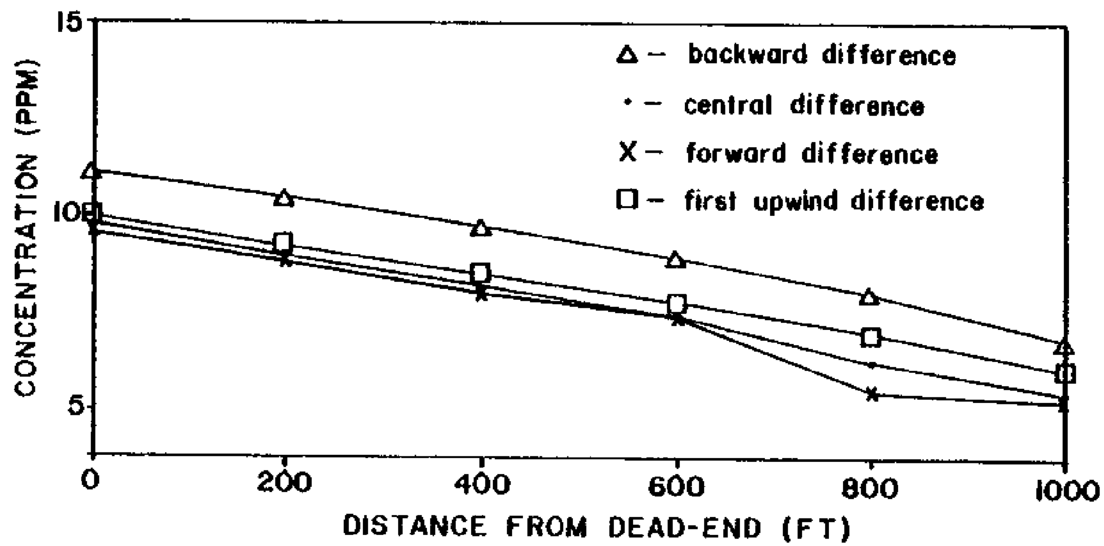


Figure 3.17 - Low Tide Concentration Profiles for Various Analog Methods.

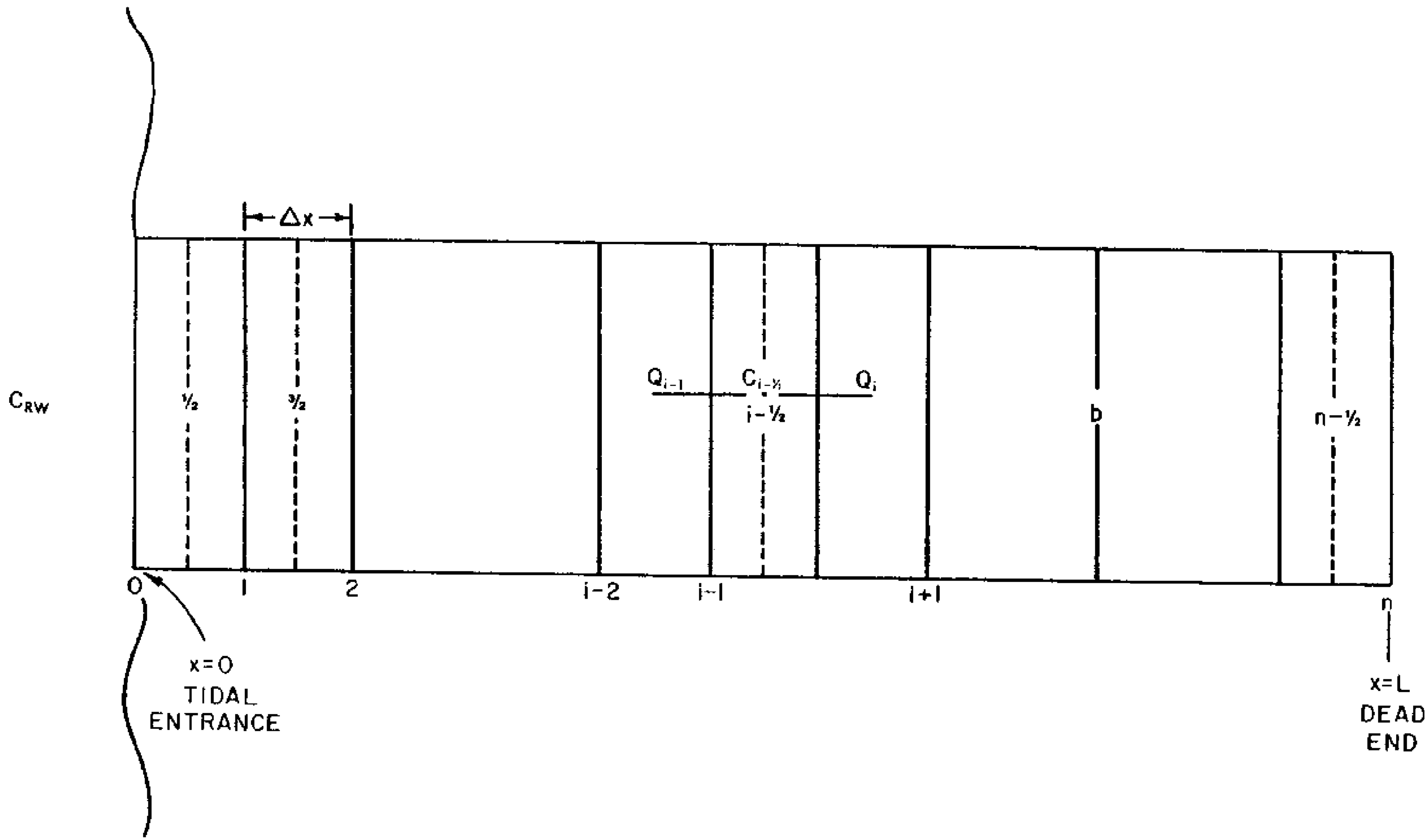


Figure 3.18 - Schematic Representation of Second Upwind Difference Method.

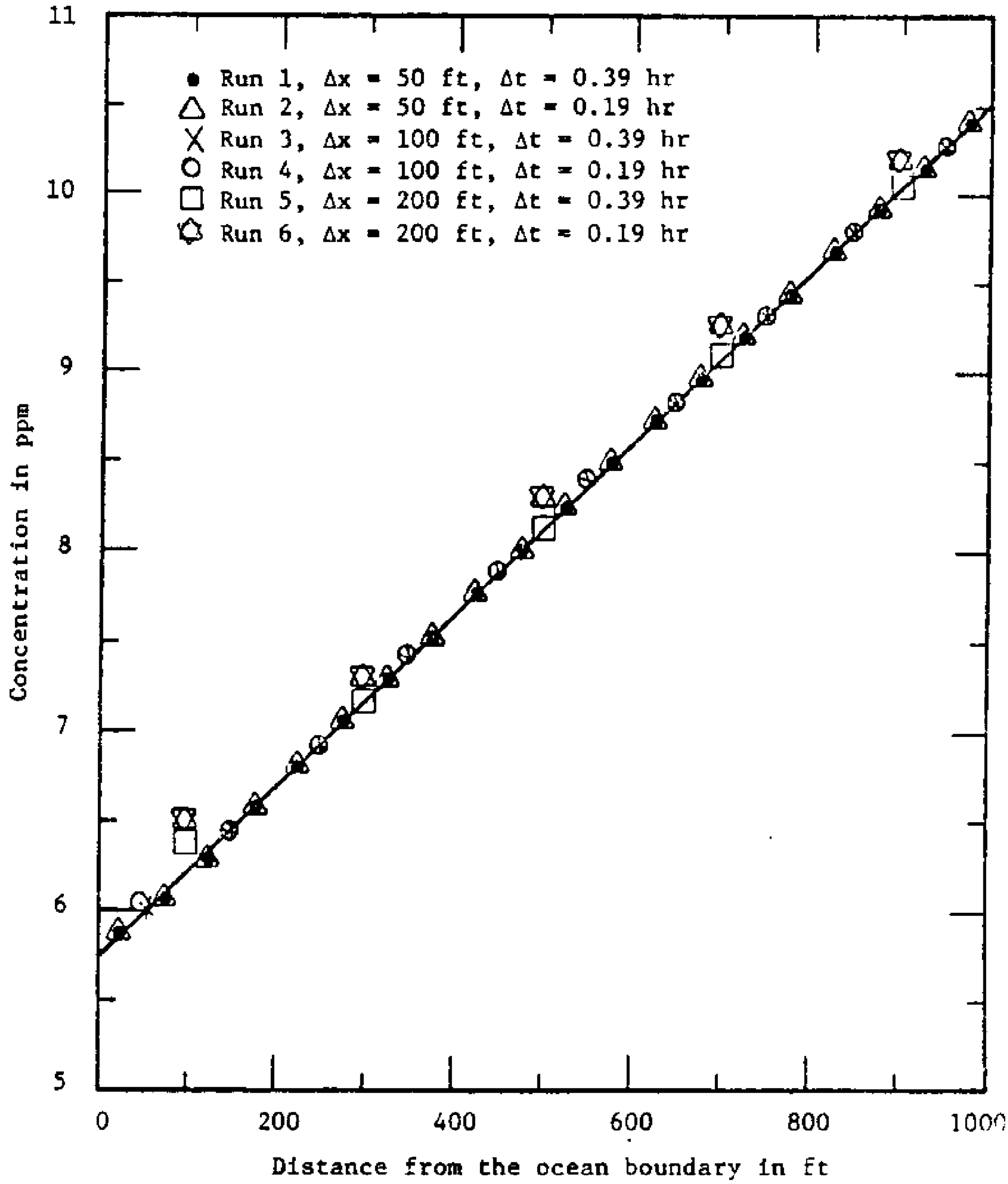


Figure 3.19 - Low Tide Concentration Profiles for Various Δx and Δt - Second Upwind Difference Method.

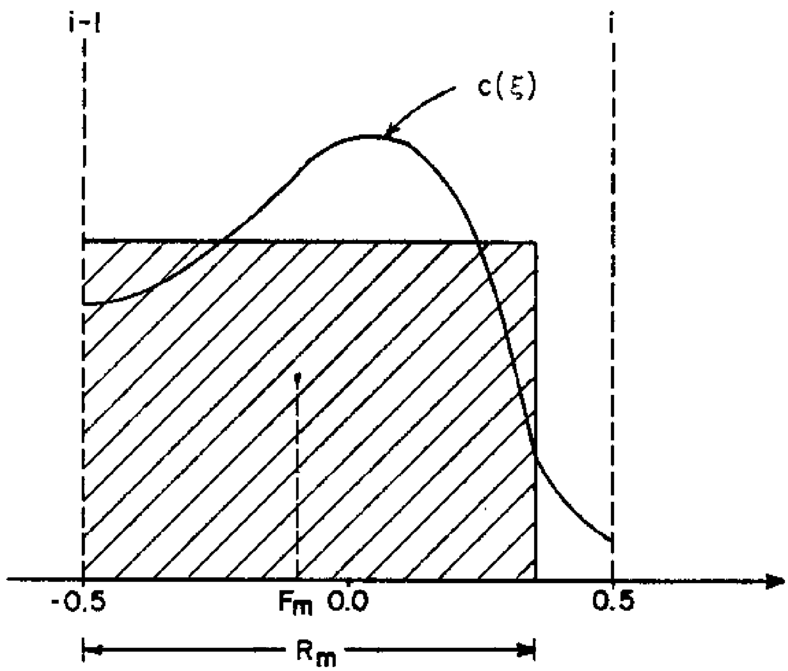


Figure 3.20- Rectangular Distribution Approximation to Actual Distribution.

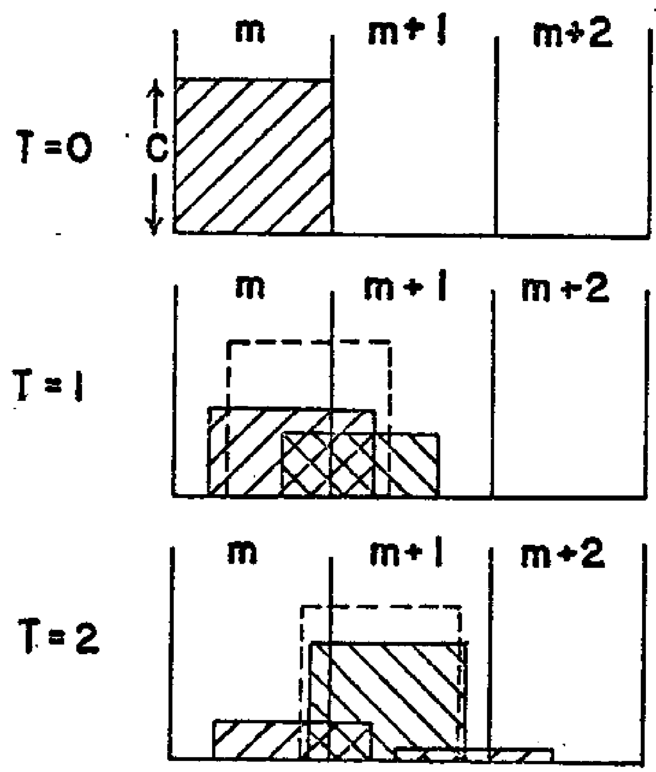


Figure 3.21 - Schematic Representation of Conservation of First Moment.

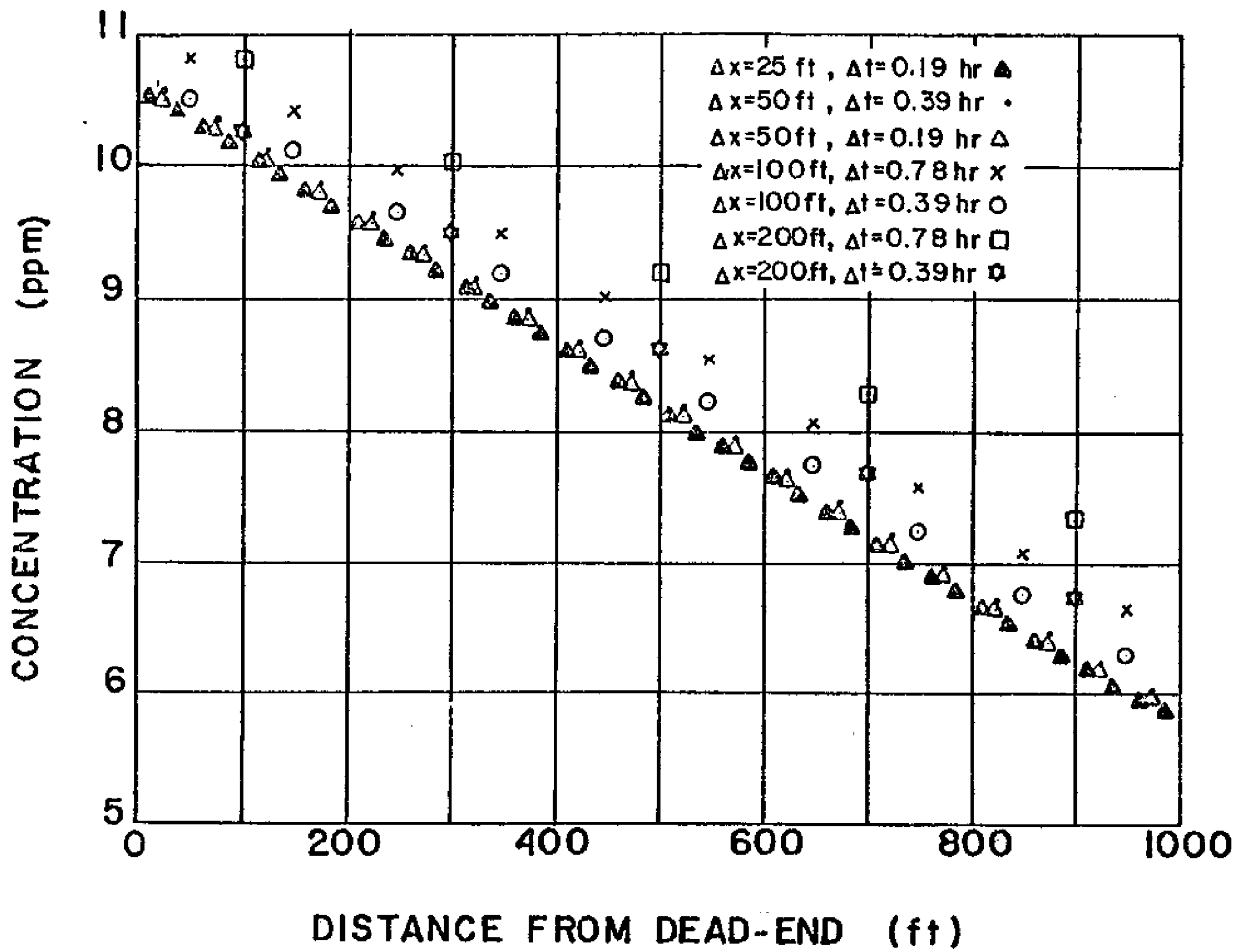


Figure 3.22 - Low Tide Concentration Profiles for Various Δx and Δt - Method of Second Moments.

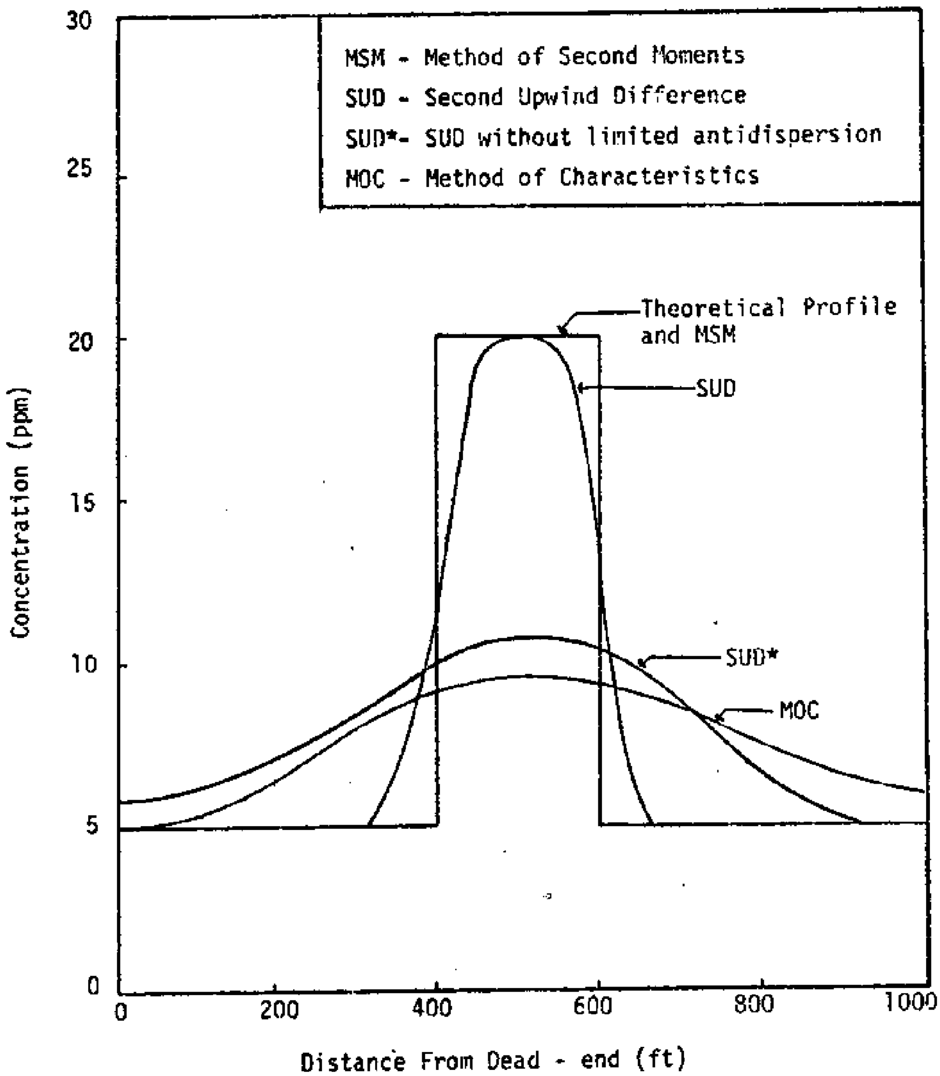


Figure 3.23 - Comparison of Techniques' Accuracy in Modeling Pure Convection.

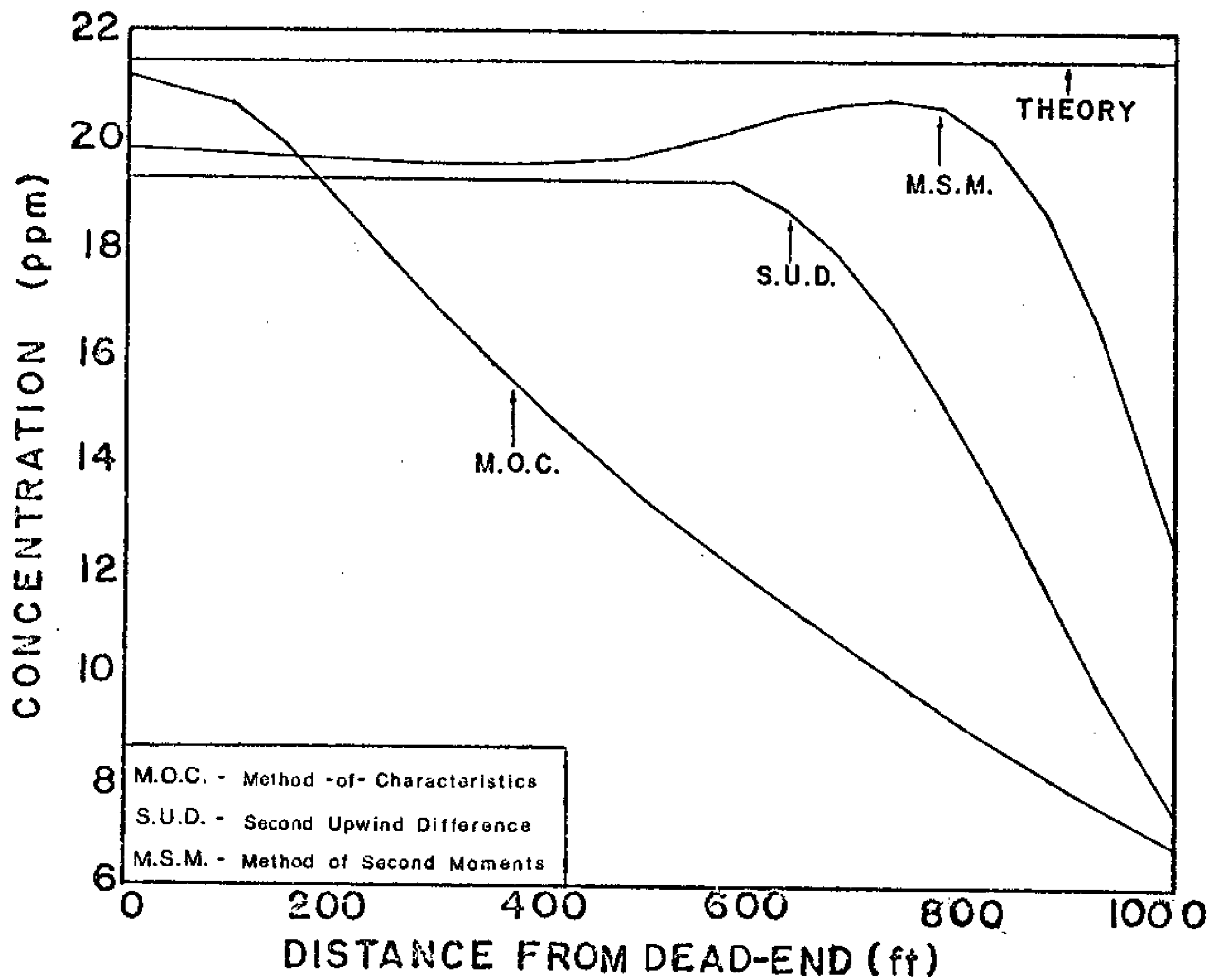


Figure 3.24 - Comparison of Techniques' Accuracy in Conserving Mass.

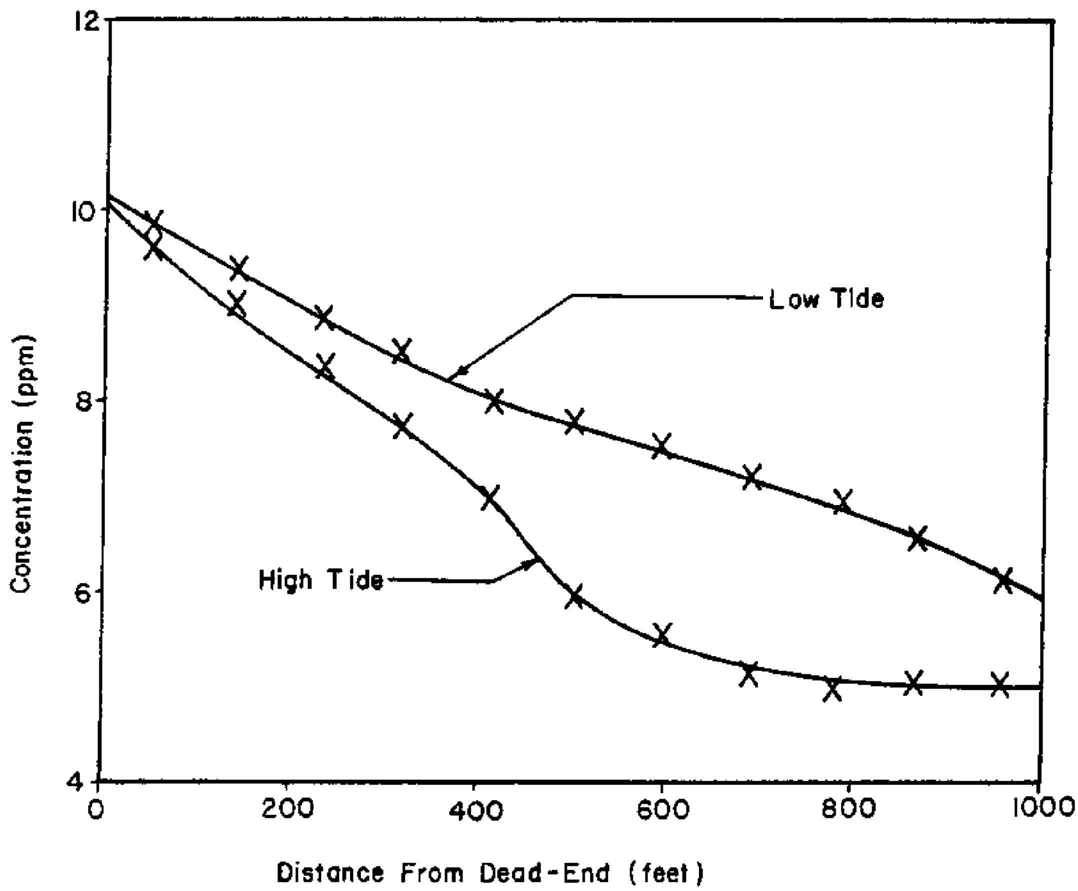


Figure 3.25 - Concentration Profiles for Second Test Canal Network - Second Upwind Difference Method.

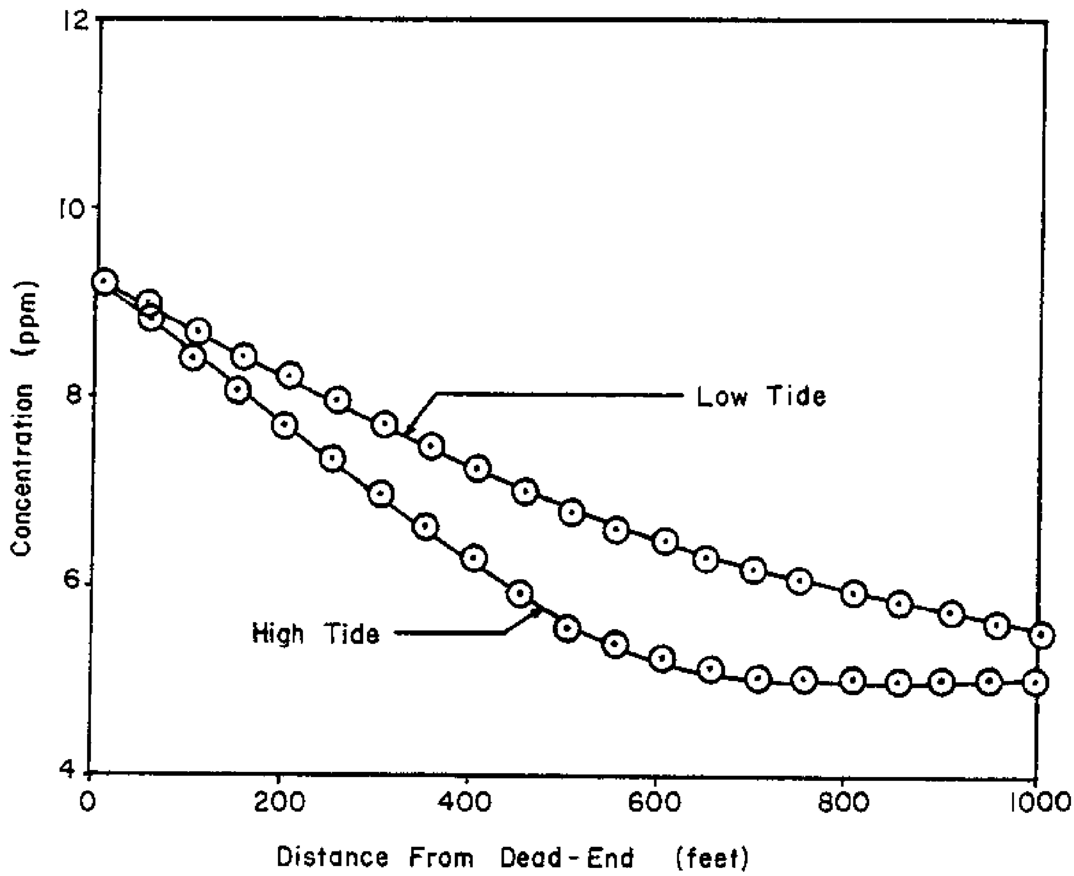


Figure 3.26 - Concentration Profiles for Second Test Canal Network - Method of Second Moments.

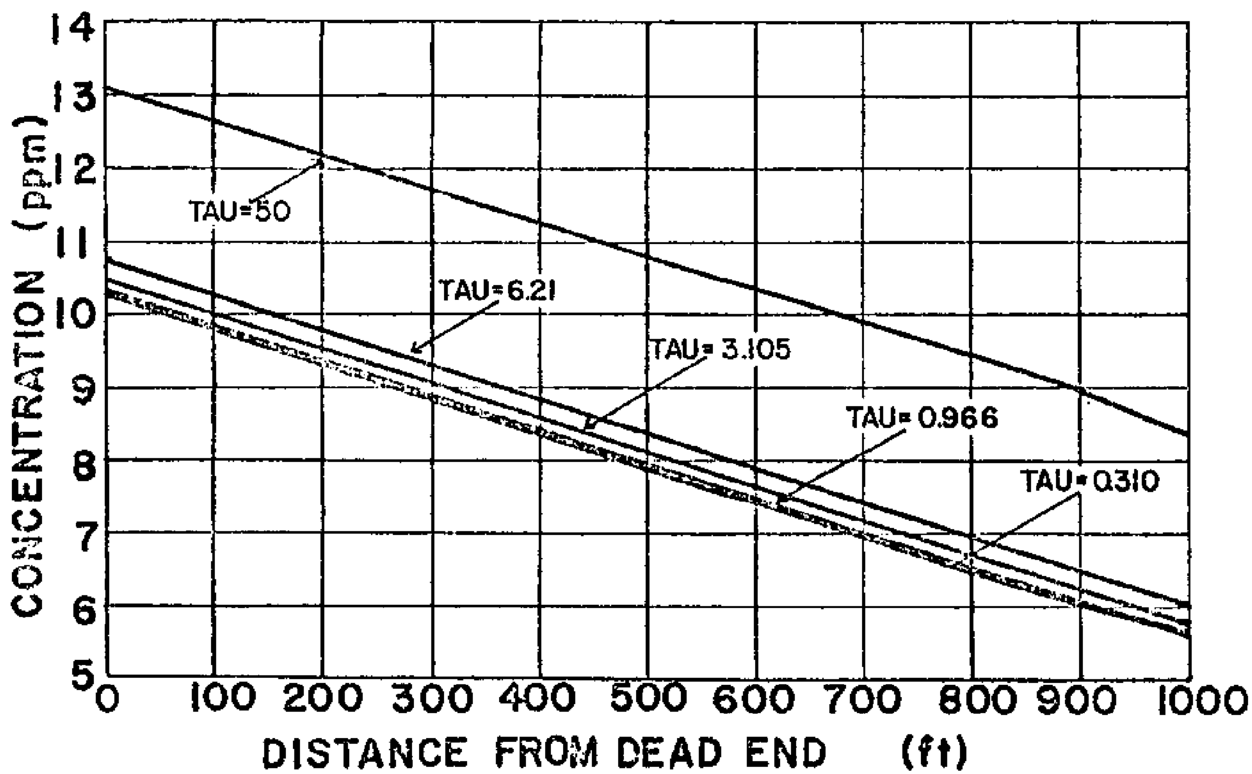


Figure 3.27 - Variability of Tidal Entrance Time Decay Coefficient, τ.

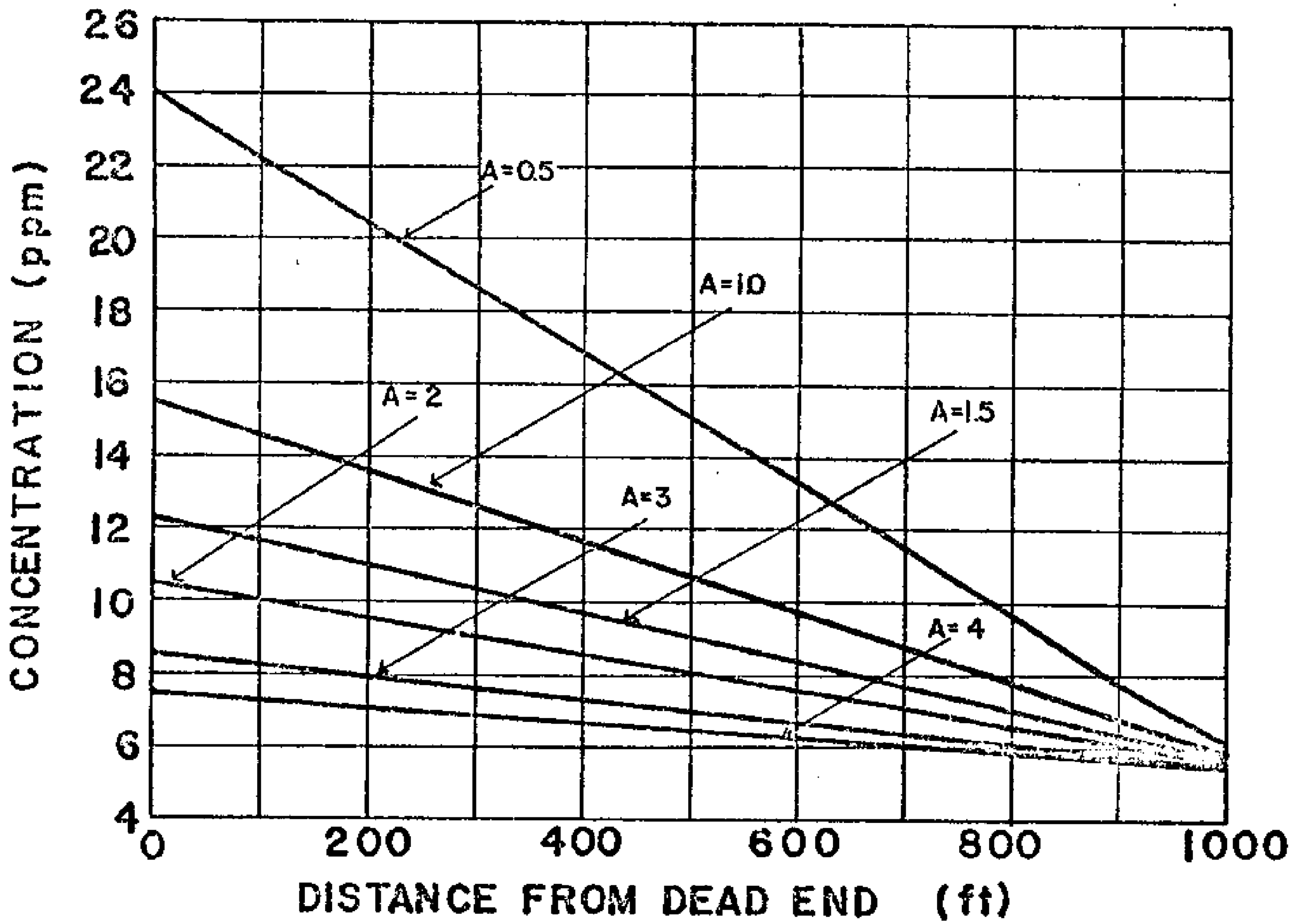


Figure 3.28 - Variability of Tidal Amplitude, a.

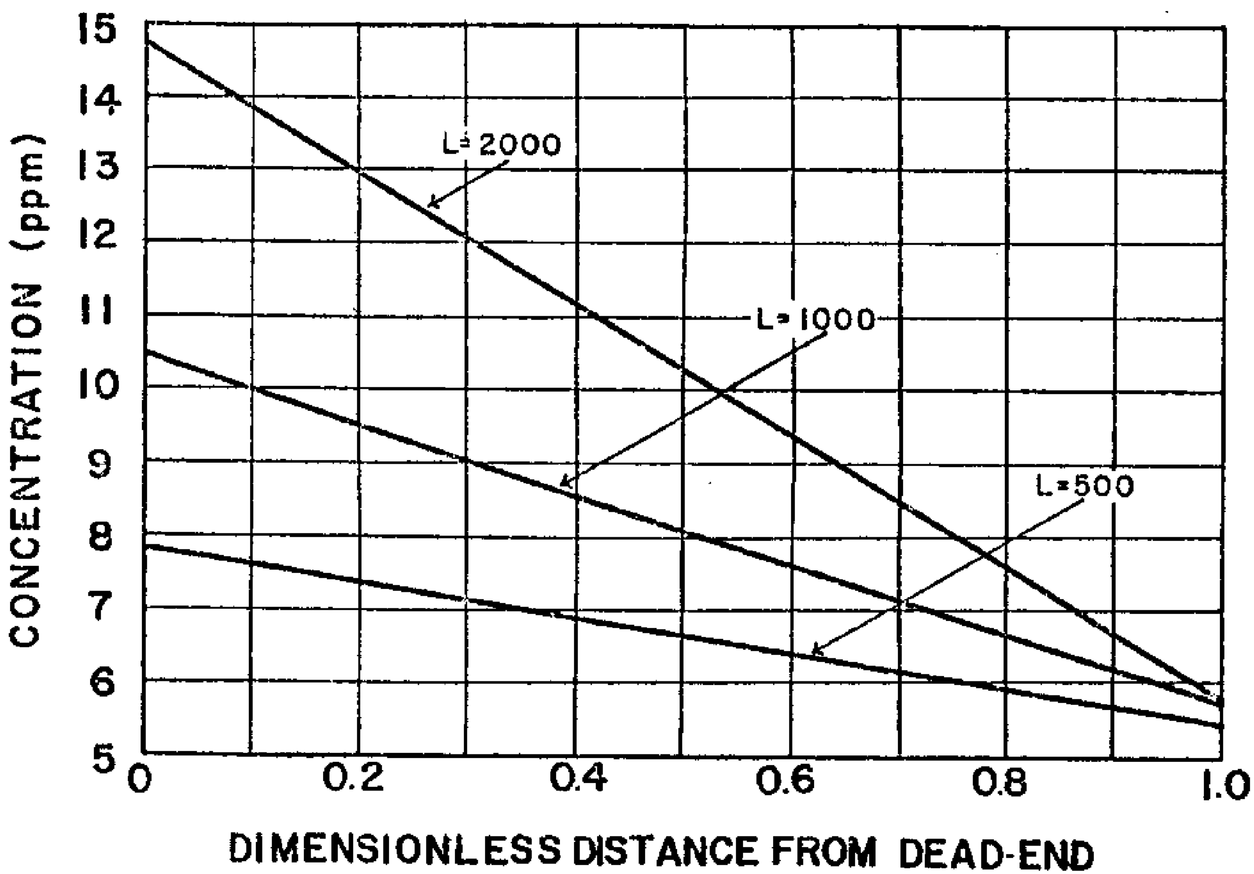


Figure 3.29 - Variability of Canal Length, L

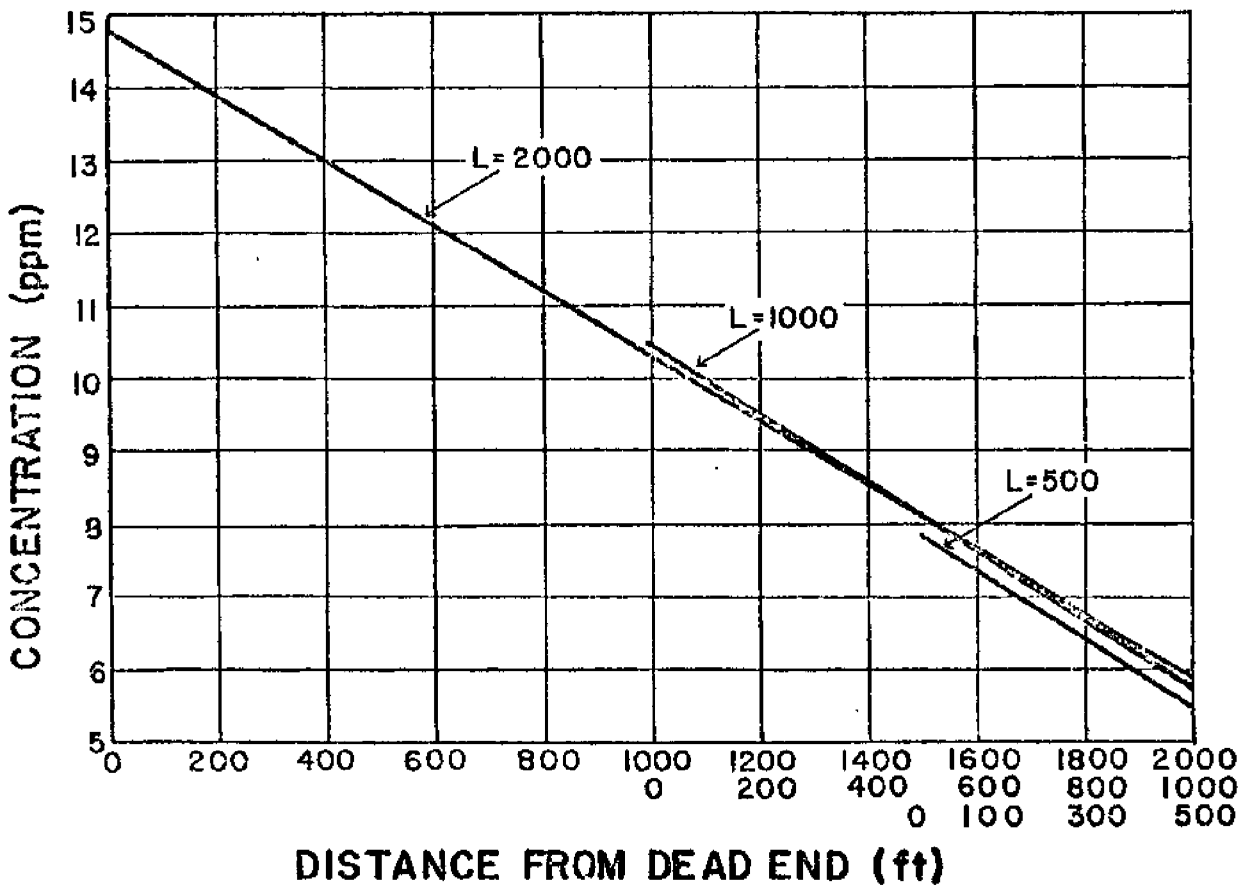


Figure 3.30 - Variability of Dimensionless Canal Length.

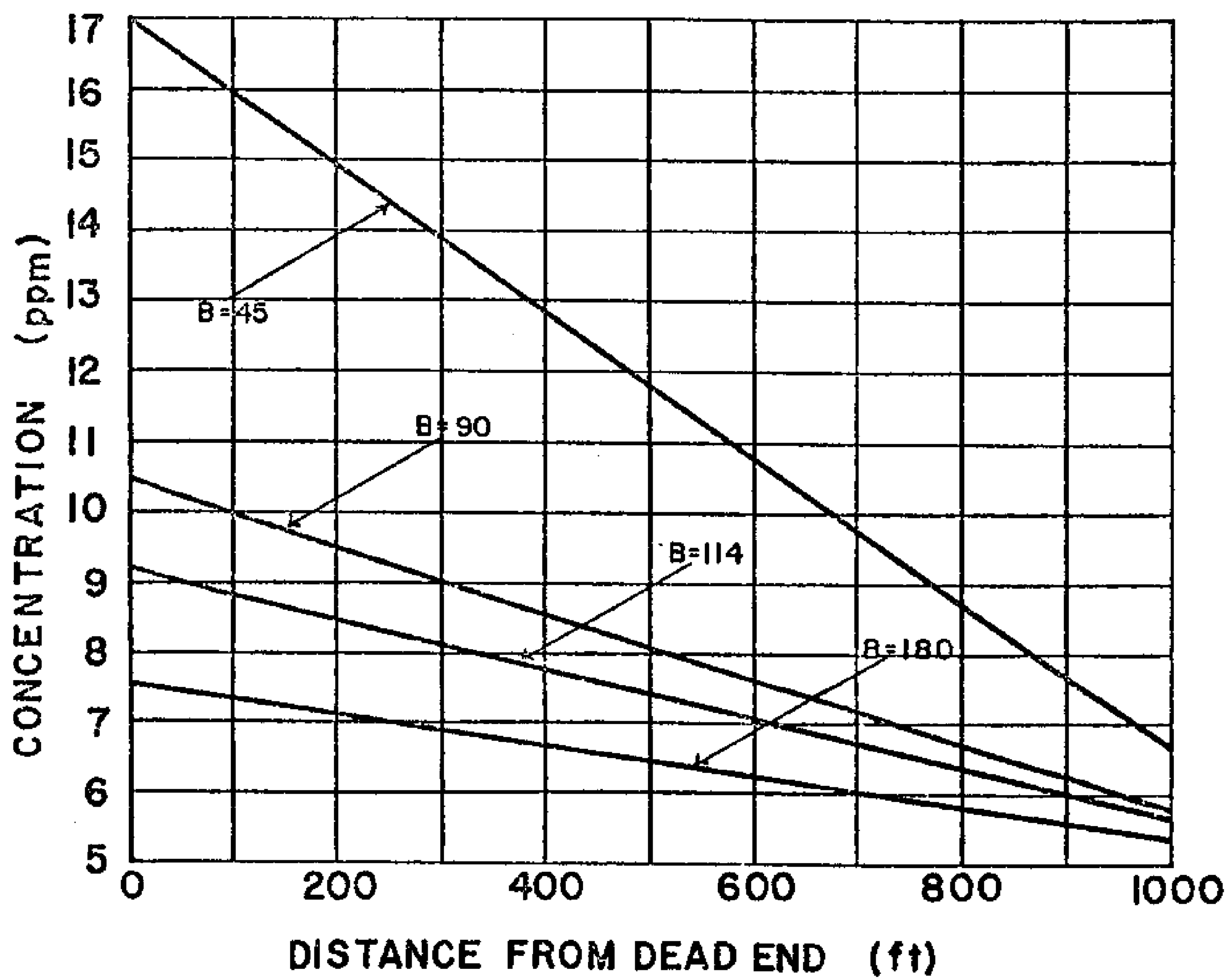


Figure 3.31 - Variability of Bottom Width, b.

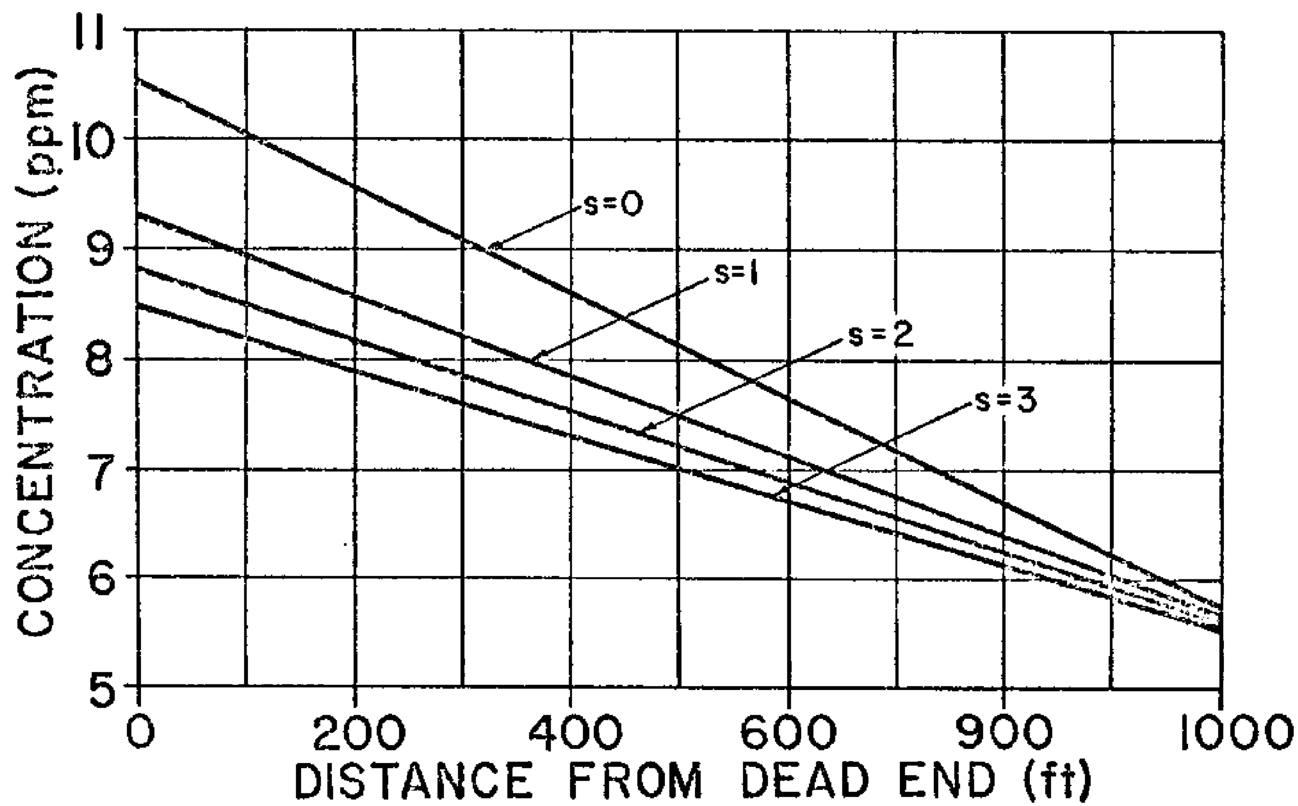


Figure 3.32 - Variability of Inverse Side Slope, s.

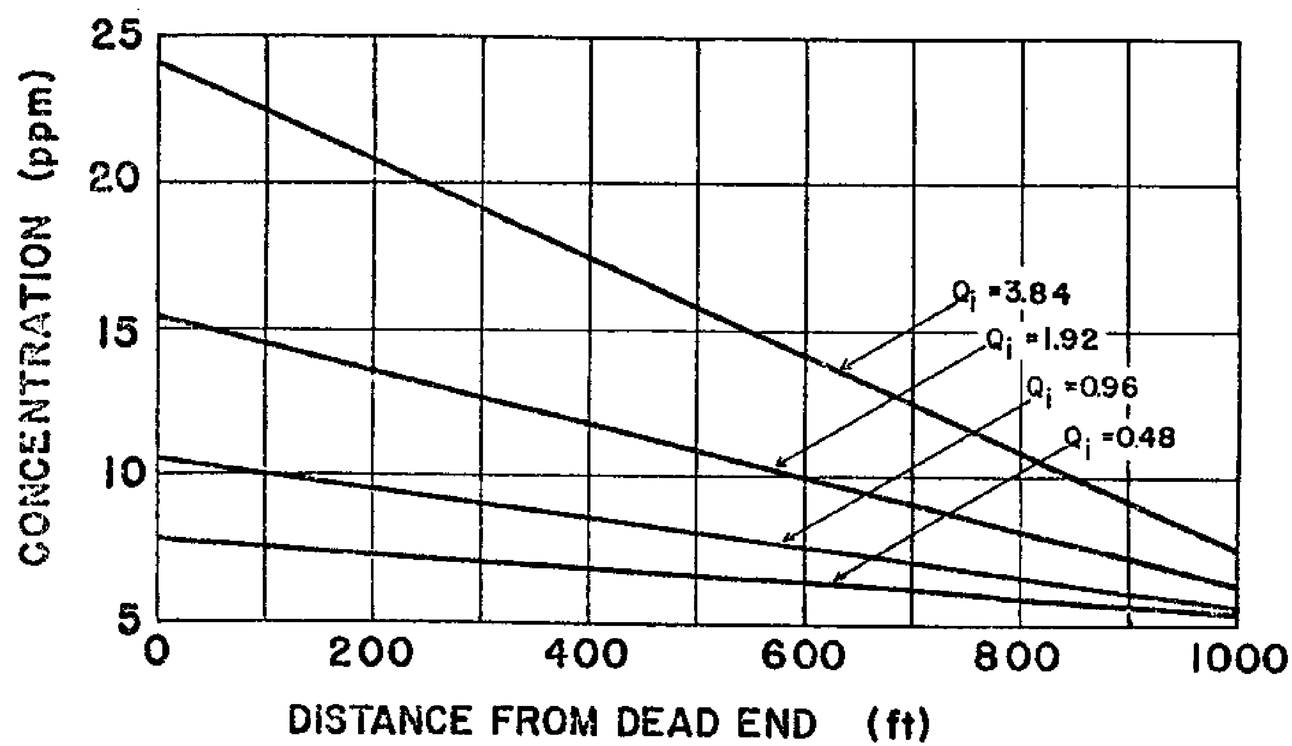


Figure 3.33 - Variability of Lateral Inflow Rate, q_I.

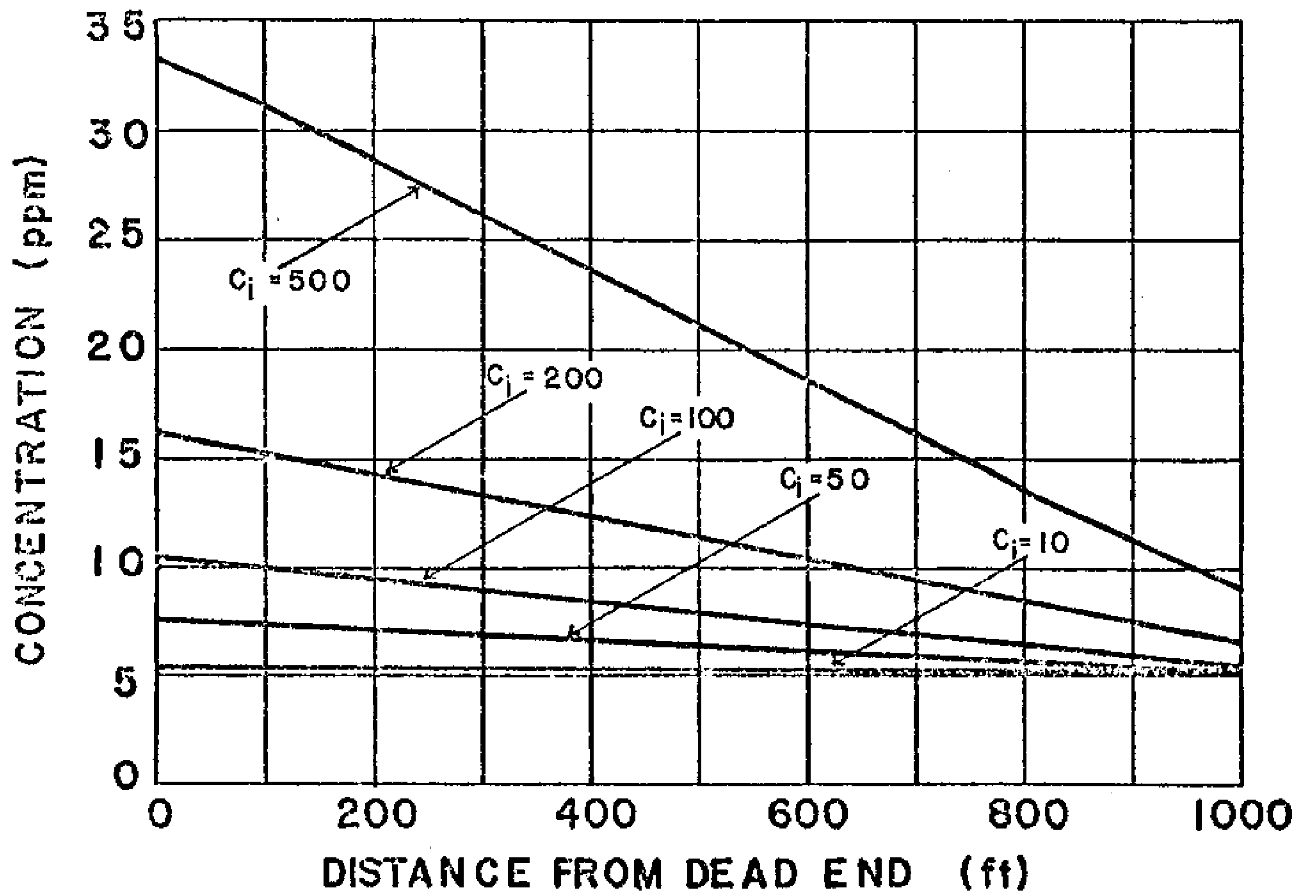


Figure 3.34 - Variability of Lateral Inflow Concentration, c_I.

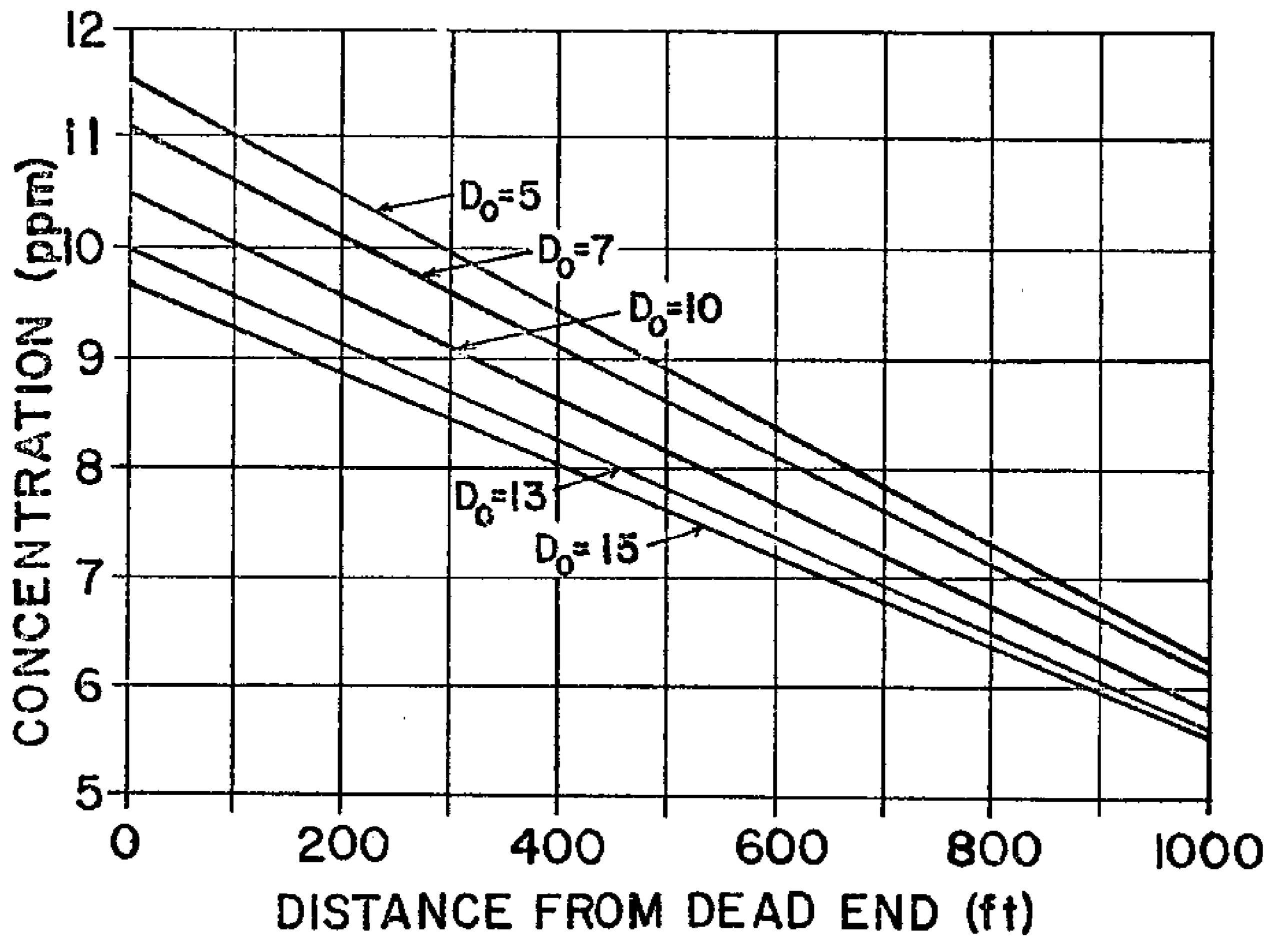


Figure 3.35 - Variability of Mean Tidal Depth, d_o.

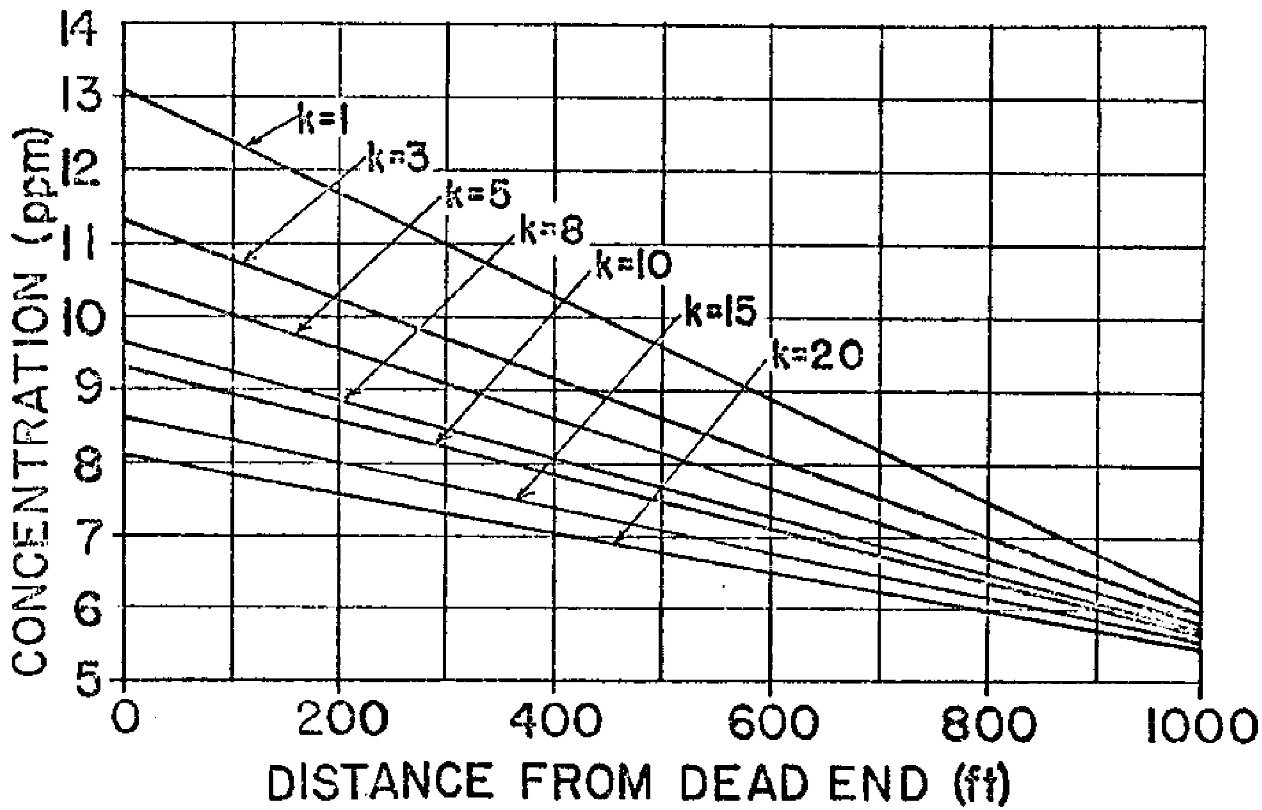


Figure 3.36 - Variability of Nikuradse's Equivalent Sand Roughness, k.

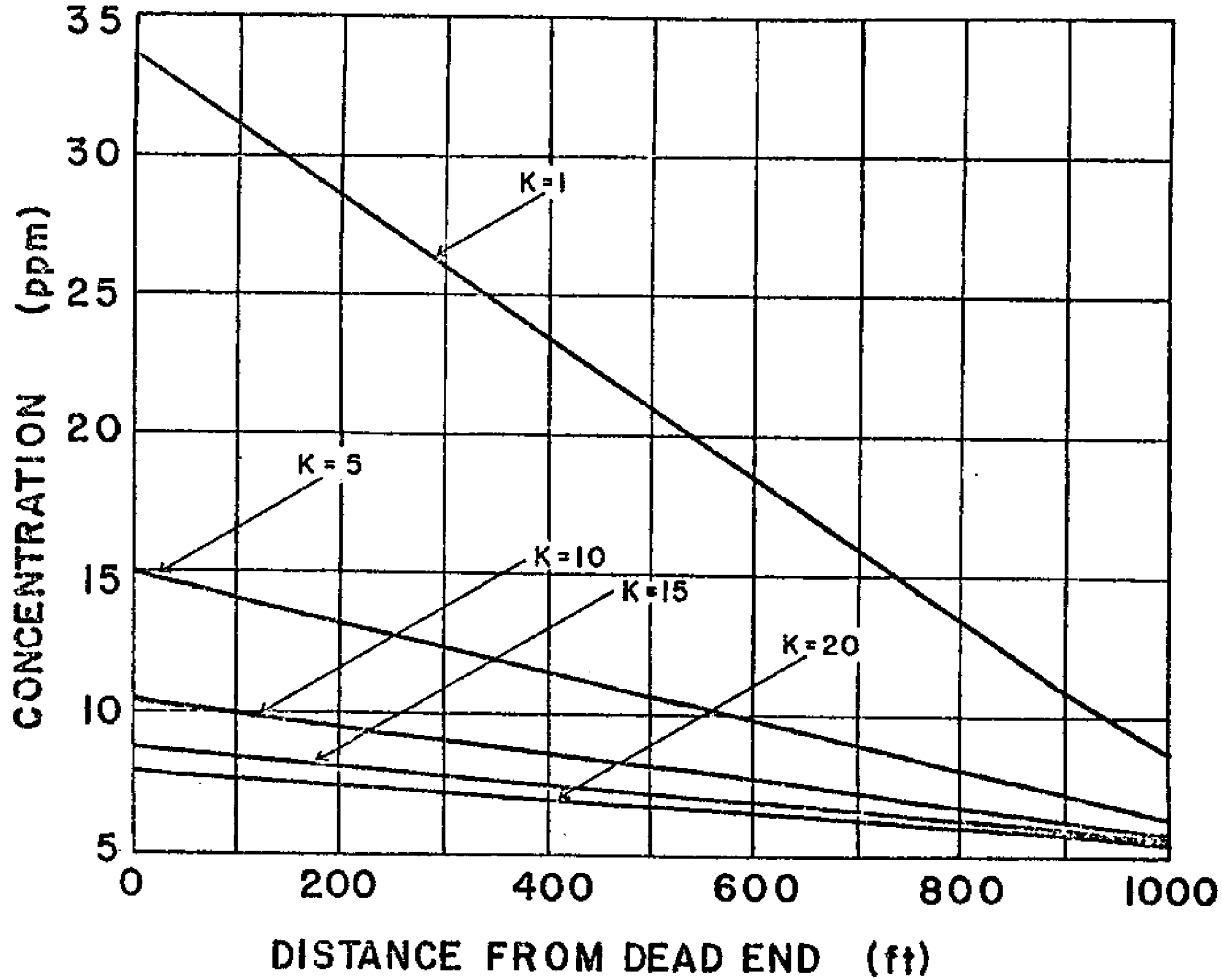


Figure 3.37 - Variability of Dimensionless Dispersion Coefficient, K.

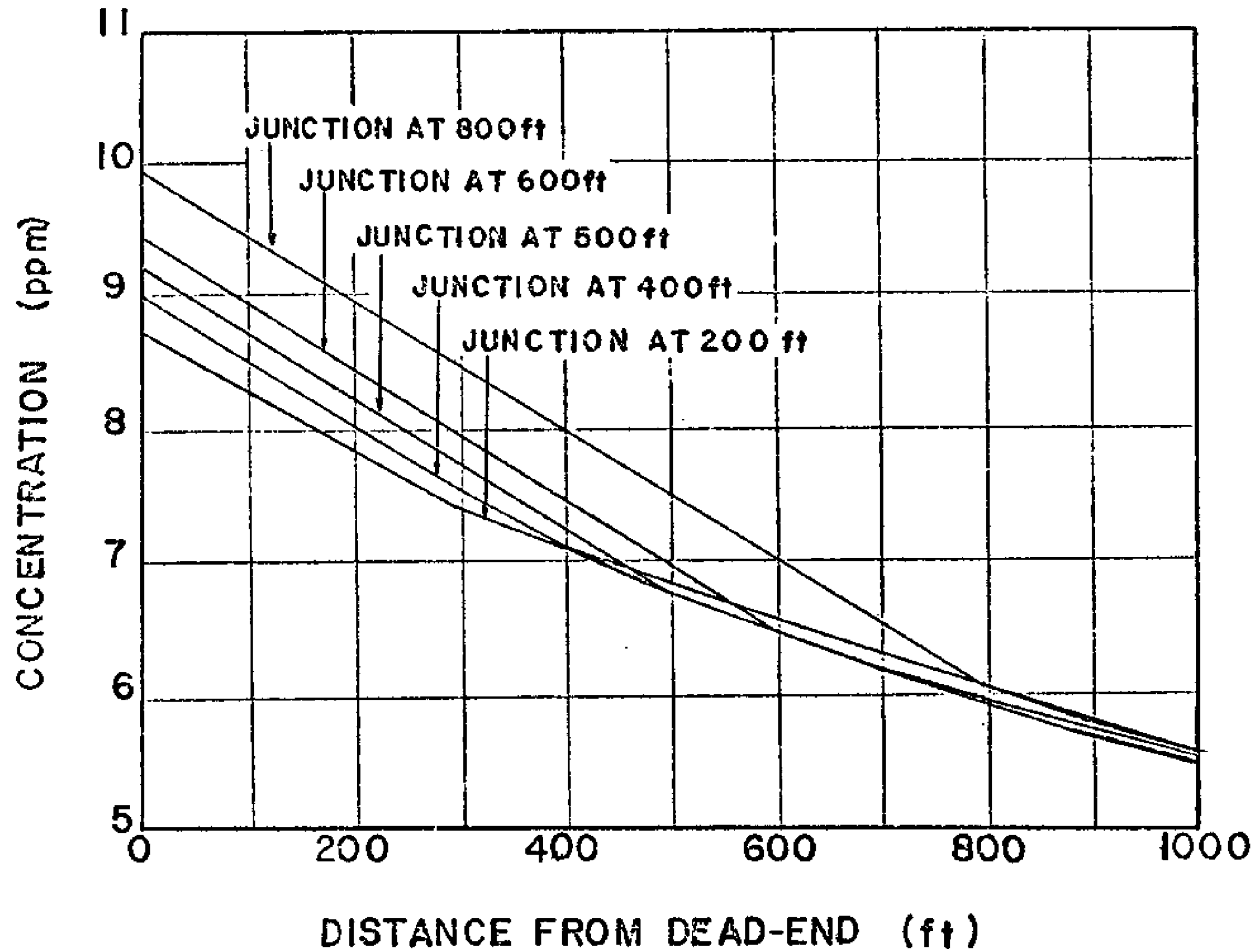


Figure 3.38 - Variability of Low Tide Concentration Profiles for Various Branch Canal Locations.

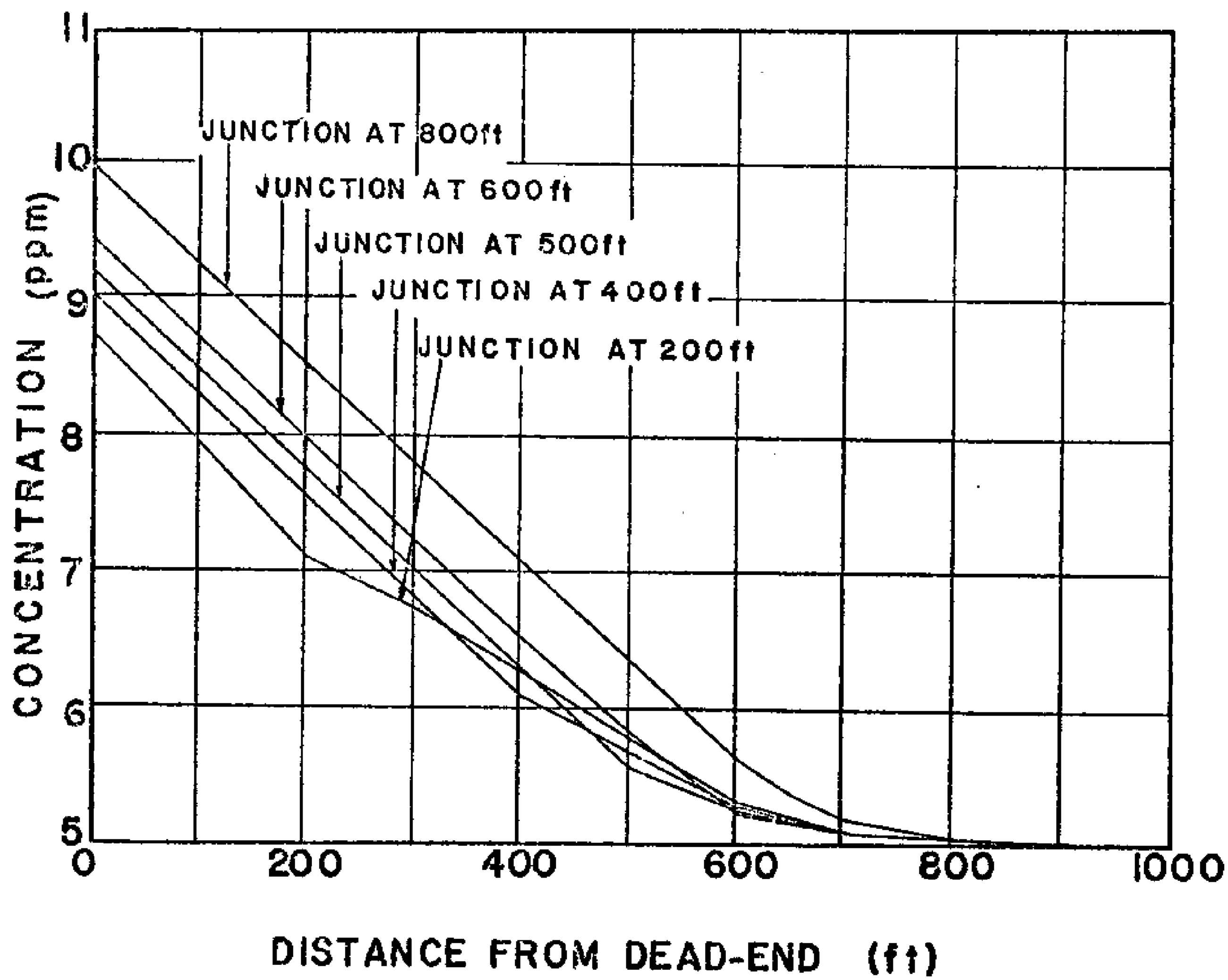


Figure 3.39 - Variability of High Tide Concentration Profiles for Various Branch Canal Locations.

Table 3.1 - 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 Gulf of Mexico	2 - 5 ft 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 3.2 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[1]	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	1.18×10^{-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 3.3 - 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
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 3.4 - Standard Data Set for Second Test Canal.

PARAMETER	VALUE
Length of Reach No. 1, L_1	450 ft
Length of Reach No. 2, L_2	500 ft
Length of Reach No. 3, L_3	450 ft
Length of Junction, Δx_j	100 ft
Length of Junction, Δy_j	90 ft
All other parameters are as for Table 3.1.	

Table 3.5 - Comparison of Numerical Techniques.

TECHNIQUE	ADVANTAGES	DISADVANTAGES
Finite-element, finite-difference	Ease of formulation, can be conservative.	Suffers from excessive numerical dispersion unless economic choices for Δx and Δt are made.
Method-of-Characteristics	Ease of formulation; very economic.	Suffers from inaccuracies in the interpolation between nodal values; nonconservative.
Hybrid	Ease of formulation; speed of solution; ease of variability studies of nongeometric parameters.	Ties up too much hardware for canal systems with more than 20 nodes; nonconservative.
Second upwind differencing with flux-corrected transport	Accurate; reasonably economic; conservative; transportive.	Complex formulation; aritificial physical form.
Method of second moments	Very accurate; reasonably economic; intuitive physical meaning; conservative; transportive.	Suffers some from round-off errors in square root term.

CHAPTER 4

THREE-DIMENSIONAL HYDRODYNAMICS IN CANAL NETWORKS

4.1 Introduction

The one-dimensional form of the hydrodynamics and transport equation was fine for investigating the numerical dispersion present in solution schemes, their accuracy and stability, and to observe the effects of variations in geometric parameters. However, the flow in a canal network cannot be reduced to this level. Even assuming a logarithmic velocity profile, when wind and salinity effects can be ignored, the excursion of a fluid particle on the surface is greater than that calculated from the mean velocity alone.

When dye, or another source load, is dumped into a canal network, it is rarely mixed as an instantaneous plane source, even with the forced mixing done on a number of field trips [Morris, 1978]. The dye cloud tends to remain in the top half of the cross-sectional area, and not be mixed near the banks. The length scale associated with the so-called 'convective period' for the dye is usually on the order of the length of the canal network itself, and thus the material will tend to be convected with the velocities in the upper layers of water, with some mixing to the lower layers. This length is defined as the distance traveled with the mean velocity before the one-dimensional mass-transport equation can be assumed to apply to quasi-steady unidirectional flow. The one-dimensional equation, then, can only be assumed to be applicable when flow reversals are not present.

The equation for this length scale, L, was given by Fischer [1967] as,

$$L > 1.8 \frac{\ell^2}{R} \frac{u}{u^*} \tag{4.1}$$

where

ℓ = distance from point of maximum velocity to the farthest bank, (L).

Now

$$u^* = \sqrt{gRS_f} \tag{4.2}$$

where

S_f = slope of energy gradient line, (dimensionless).

From Manning's equation in the form,

$$u = \frac{8.25\sqrt{g}}{k^{1/6}} R^{2/3} S_f^{1/2} \tag{4.3}$$

u* can be written from Equation (4.2) as,

$$u^* = \frac{u}{8.25} \left(\frac{k}{R}\right)^{1/6} \tag{4.4}$$

In Floridian canals, k is of the same order as R, and as the ratio is raised to the 1/6 power, it may be assumed that,

$$\frac{u}{u^*} \approx 8.25 \tag{4.5}$$

Thus, for a typical canal,

$$L > \frac{1.8 \cdot 50^2 \cdot 8.25}{10} = 3{,}700 \tag{4.6}$$

which is on the order of the length of the canal, and over which, the one-dimensional mass-transport equation does not apply.

Consider for example, a wind blowing over the canal network in the positive x-direction. Then the excursion of the top layers will be even greater than for an ebb tide alone with a resulting logarithmic velocity distribution. Added to this, there is a reversal theoretically at the one-third depth, which convects any dye in the lower layer upstream. The combination of this tidal flow and wind circulation produces a dye cloud with a large leading concentration gradient and a long tail, as diffusion occurs not only longitudinally in the top layer of water, but also from the reversal layer to the top layer (Figure 4.1).

This type of dye concentration distribution was frequently measured in a number of canal networks, such as the 57 Acres Site, in which the wind circulation was a major factor, and cannot be predicted by any one-dimensional theory. In the site on the Loxahatchee River, and also in two canals investigated by McKeehan [1975], gravity flows due to saline density gradients are an important factor, and according to McKeehan, the dominant factor for his two Gulf Coast canals. In these cases, the canals are hydraulically connected to a tidal estuary with a large river freshwater flow. In this system, a saltwater wedge enters the canal during the flood tide and receeds during the ebb tide in which the freshwater river flow flushes most of the remaining saltwater wedge in the canal leaving a salt dome. It is the exception rather than the rule to measure Gaussian shaped concentration profiles, and a two or three-dimensional theory must be used to explain the physical processes.

Another important phenomenon in these canal networks is secondary currents. This is one of the major mechanisms by which the anoxic water in the benthic layer at the bed may be transferred to the surface where it can be resupplied with oxygen. To include this effect, the wind circulation, density flows, and tidal flow, requires a three-dimensional model.

The assumption that the water surface can be considered as a horizontal plane, rising and falling through the tidal range, with the tidal frequency, has already been introduced in developing the one-dimensional model. This effect was, of course, measured for a three-dimensional hydrodynamic system, and it therefore seems justifiable to extend this assumption to a three-dimensional model.

The assumption can be supported in theory if the slope of the water surface due to wind set-up alone in a dead-end canal is calculated. Bretschneider [1966], using Saville's work [1953], proposed the following equation for the slope of the water surface under these conditions.

$$\frac{\partial \eta}{\partial x} = \frac{3.3 \cdot 10^{-6} w_s^2}{g(d_o + \eta)} \tag{4.7}$$

where

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

w_s = wind speed, (L/T)

Typically in Floridian canals,

$$\frac{\partial \eta}{\partial x} = \frac{3.3 \cdot 10^{-6} \cdot 15^2}{32.13 \cdot 10} = 2.3 \cdot 10^{-6} \tag{4.8}$$

which is too small to measure in most networks.

In this chapter, the major hydrodynamic factors such as the tidal flow, wind induced circulation, density currents and secondary currents are investigated, and closed form expressions derived for their independent effects. It will be assumed that these separate velocity distributions may be superimposed to obtain the network velocity field. This assumption will be supported by field data.

Finally, it remains to define a coordinate system (Figure 4.2). Using the definitions of Section 3.1.2, the x-axis (longitudinal) is in the direction of the ebb tide in each reach, the y-axis (lateral) is from left bank to the right bank when facing in the positive x-direction, the z-axis is vertically upward from the bed of each reach, which is assumed to be a horizontal plane.

4.2 Tidal Velocities

When wind and salinity gradient effects are negligible, the flow is usually found to be logarithmic. The vertical velocity profile based on the volume of the tidal prism, V, upstream of the cross-section of interest, is obtained from Nikuradse's [1933] logarithmic profile as,

$$u(z) = \frac{1}{A}\frac{dV}{dt}\frac{\ln(29.73z/k)}{\ln(29.73d/ke)} \qquad (4.9)$$

where

$$e = \text{exponential constant} = 2.718.$$

This form of the equation is satisfactory for a vertically layered model because the mean velocity in the bottom layer, u_b, from $z = o$ to $z = d$, where d_b is the thickness of the bottom layer is given by,

$$u_b = \frac{1}{d_b}\int_o^{d_b} u(z)\,dz \qquad (4.10)$$

Integrating,

$$u_b = \frac{1}{A}\frac{dV}{dt}\frac{\ln(29.73d_b/ke)}{\ln(29.73d/ke)} \qquad (4.11)$$

as $z \ln z \to o$ as $z \to o$.

Similarly, the mean velocity in any other layer, u_ℓ, from $z = d_1$ to $z = d_2$ is given by,

$$u_\ell = \frac{1}{A}\frac{dV}{dt}\frac{(d_2 \ln(29.73d_2/ke) - d_1 \ln(29.73d_1/ke))}{(d_2 - d_1) \ln(29.73d/ke)} \qquad (4.12)$$

In a layered model, and particularly one in which the velocities are constant in each layer, it is usual to neglect the no-slip boundary conditions at the bed and the sides of the channel. This is because the thickness of the layer associated with large velocity gradients near the solid boundaries is very small compared to the smallest dimension of a computational cell, a cell being formed by the intersections of planes defining the lateral and vertical layers, and of length Δx.

Instead to allow for vertical and lateral variations in the flow field, the logarithmic velocity defined for any cell was based on the mean depth of the projectors of the horizontal area onto the bed (Figure 4.3). This technique gives a lateral variation to the velocity field for a trapezoidal cross-section.

To give a vertical variation, it was assumed that each cell increases its cross-sectional area by a factor proportional to its mean velocity vector. The constraint placed on this technique is that the cells in a particular layer all expand or contract vertically at the same rate allowing nonuniform expansion only in the lateral direction. The sum of all the expansions or contractions of the cells then equals the change in the total cross-sectional area of the flow in one time interval, Δt.

4.3 Wind Induced Circulation

Many researchers have witnessed and measured flow reversals due to a wind blowing over a waterbody contained in a semienclosed basin [Hellstrom, 1941; Keulegan, 1951; Wu, 1969], and have derived closed form functions or empirical formulae describing the vertical velocity profile. The turbulent momentum equation in the longitudinal x-direction can be written without density terms,

$$\frac{du}{dt} = g\frac{\partial\eta}{\partial x} + fv + \frac{\partial}{\partial x}\left(\frac{\tau_{xx}}{\rho}\right) + \frac{\partial}{\partial y}\left(\frac{\tau_{xy}}{\rho}\right) + \frac{\partial}{\partial z}\left(\frac{\tau_{xz}}{\rho}\right) - \frac{1}{\rho}\frac{\partial Pa}{\partial x} \tag{4.13}$$

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, z directions with respect to x-direction, (M/LT^2)

ρ = density, (M/L^3)

Pa = atmospheric pressure, (M/LT^2).

Assuming quasi-steady flow, which is a good assumption as the tidal ranges are very small in Floridian canals, that the atmospheric pressure, Pa, is uniform over the canal network (on the order of a few square miles), that the longitudinal and lateral gradients of the shear stress terms are small, and neglecting the Coriolis effect,

$$g\frac{\partial\eta}{\partial x} = \frac{\partial}{\partial z}\left(\frac{\tau_{xz}}{\rho}\right) \tag{4.14}$$

Considering a rectangular prismatic dead-end canal and assuming that the water surface slope $\partial\eta/\partial x$ is constant, then

$$\frac{\tau_{xz}}{\rho} = gz\frac{\partial\eta}{\partial x} + a_o \tag{4.15}$$

where

a_o = constant of integration.

The classical way to write the wind shear stress is,

$$\tau_{xs} = K_w \rho \, w_s^2 \cos\theta \qquad (4.16a)$$

$$\tau_{ys} = K_w \rho \, w_s^2 \sin\theta \qquad (4.16b)$$

where

K_w = drag coefficient, (dimensionless)

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

w_s = wind speed, (L/T)

τ_{xs}, τ_{ys} = surface wind shear stresses in x and y- directions respectively, (M/LT^2)

However, this is assuming a resultant shear stress, τ_x, of the two components, τ_{xs} and τ_{ys}. In a canal reach, the lateral shear stress is very much smaller than the longitudinal, and may be ignored. Hence here,

$$\tau_{xs} = K_w \rho \, w_s^2 \cos\theta \, |\cos\theta| \qquad (4.17a)$$

$$\tau_{ys} \simeq 0 \qquad (4.17b)$$

The form of the wind drag coefficient, K_w, has been studied by a number of people, notably Van Dorn [1953] and Wu [1969]. Van Dorn's work was considered the standard until Wu did a more thorough investigation incorporating Van Dorn's results. He put forward the following relationship (converting from metric to English units) which will be used in this model,

$$K_w = 0.6 \times 10^{-6} \, w_s^{1/2} \qquad \text{for} \qquad w_s < 49.2 \text{ ft/sec.} \qquad (4.18a)$$

$$K_w = 3.1 \times 10^{-6} \qquad \text{for} \qquad w_s > 49.2 \text{ ft/sec.} \qquad (4.18b)$$

With the above boundary condition (Equation (4.17a)), at $z = d$, the depth of water in the reach, the constant a_o in Equation (4.15) can be evaluated, and,

$$\frac{\tau_{xz}}{\rho} = K_w\, w_s^2 \cos\theta\, |\cos\theta| + g(z - d)\,\frac{\partial\eta}{\partial x} \tag{4.19}$$

The z-component of the turbulent shear stress term can be written

$$\tau_{xz} = \left(\nu\frac{\partial u}{\partial z} - \overline{u'w'}\right) \tag{4.20}$$

where

ν = kinematic viscosity, (L^2/T)

u', w' = turbulent fluctuation from turbulent time mean velocities in x and z directions respectively, (L/T)

$(\overline{\ \ })$ = turbulent time mean value.

Using the laminar analogy, which is the first term on the left hand side of Equation (4.20), the turbulent shear stress can be written in terms of a vertical eddy viscosity coefficient, or vertical momentum transfer coefficient, N_z, as,

$$\tau_{xz} = \rho N_z \frac{\partial u}{\partial z} \tag{4.21}$$

Substituting Equation (4.21) into Equation (4.19),

$$N_z \frac{\partial u}{\partial z} = K_w\, w_s^2 \cos\theta\, |\cos\theta| + g(z - d)\,\frac{\partial\eta}{\partial x} \tag{4.22}$$

The next step involves making some assumption about the form of the vertical momentum transfer coefficient, N_z. As $w' = 0$ at the bed, and as $\partial u/\partial z$ is very large (infinite for a no-slip boundary condition), then from Equations (4.8) and (4.9), N_z must be very small or zero there. At the surface most investigators considered $N_z = 0$, or that $w' = 0$

[Cooper and Pearce, 1977; Madsen, 1977]. However, this is only approximately true because there is usually some vertical surface activity in almost all open water bodies. Therefore, N_z may be small but usually not zero.

The classical form of the vertical momentum transfer coefficient is a parabola of the form,

$$N_z = 0.4u* \frac{z}{d}(1 - \frac{z}{d}) \tag{4.23}$$

This form has been substantiated by a number of investigators [Johnson and Sayre, 1970; Pritchard, 1956] who measured velocity characteristics both in the laboratory and in the field. The problem arises when such a variation is incorporated into a closed-form solution for the velocity field associated only with the wind.

Many numerical modelers have considered the vertical momentum coefficient to be constant, and have achieved fairly good results [Heaps, 1972, 1974; Wang and Connor, 1975]. Some modelers have tried to go further and incorporate a vertical parabolic distribution. Leendertse and Liu [1975] did this in a layered model by allowing the coefficient to be constant in each layer. Cooper and Pearce [1977], tried to match the distribution of the coefficient using a finite element approach approximating its shape with a series of linear segments. The results were inconclusive, but more importantly, the method introduced two new coefficients instead of the original one, which must be evaluated from field measurements. From Equation (4.22) it can be seen that N_z is a function of z to the first, or higher, power. Integrating Equation (4.13) produces a logarithmic term in z which must be evaluated at z = 0, to solve for the constants of integration.

It was felt that, although assuming N_z to be constant is not strictly correct from theoretical considerations or observations, enough people have obtained adequate results from this assumption to justify its use here. Also, introducing two or more new coefficients instead of the original one, puts more stress on the accuracy of the field data. Furthermore, when the closed-form solution is derived it will be supported by field data.

Integrating Equation (4.22) again with respect to z gives,

$$u(z) \;=\; (K_w\, w_s^2 \cos\theta\; |\cos\theta|\; z + gz(\tfrac{z}{2} - d)\; \tfrac{\partial\eta}{\partial x})/N_z \qquad (4.24)$$

The final condition left to use is that there is zero net flow through any cross section,

$$\int_0^d u(z)\; dz = 0 \qquad (4.25)$$

Applying this condition to Equation (4.24) gives,

$$\frac{\partial\eta}{\partial x} \;=\; \frac{3}{2}\,\frac{K_w\, w_s^2 \cos\theta\; |\cos\theta|}{gd} \qquad (4.26)$$

and substituting back into Equation (4.24)

$$u(z) \;= \frac{1}{4}\, K_w\, w_s^2 \cos\theta\; |\cos\theta|\; z(\frac{3z}{d} - 2)/N_z \qquad (4.27)$$

This is essentially the same result arrived at by Cooper and Pearce [1977] with a change of axis and has the parabolic form shown in Figure 4.4.

Equation (4.27) can readily be modified for a trapezoidal prismatic dead-end canal to satisfy Equation (4.25), the zero net flow condition, by multiplying the function by a ratio of the top width, B, to the width of the channel at the desired elevation, giving,

$$u(z) \;= \frac{1}{4}\, \overline{B}(z)\; K_w\, w_s^2 \cos\theta\; |\cos\theta|\; z(\frac{3z}{d} - 2)/N_z \qquad (4.28)$$

where

$$\overline{B}(z) = (b + d(s_L + s_R))/(b + z(s_L + s_R)) \qquad (4.29)$$

where

b = bottom width, (L)

s_L, s_R = inverse side slopes of left and right banks respectively, (dimensionless).

For a layered model in which the velocity is integrated over a cell from $z = d_1$ to $z = d_2$, this correction factor is then based on the cross-sectional area of each layer. Hence for layer k, the wind induced vertical velocity profile is written,

$$u_k = \frac{1}{4}\,\overline{A}(k)\;K_w\,w_s^2 \cos\theta\;|\cos\theta|[d_2^2(\frac{d_2}{d} - 1) - d_1^2(\frac{d_1}{d} - 1)]$$
$$/N_z(d_2 - d_1) \qquad (4.30)$$

where

$$\overline{A}(k) = \frac{(2b + (d + d_{-1})(s_L + s_R))}{(2b + (d_1 + d_2)(S_L + S_R))}\,\frac{d - d_{-1}}{d_2 - d_1} \qquad (4.31)$$

where

d_{-1} = elevation of base of top layer, (L).

From Equation (4.28), an apparent anomaly arises. This is that as the depth increases, the magnitude of the surface velocity also increases as a linear function of the depth. This would therefore result in surface velocities that were greater than the wind speed itself if the canal were made deep enough. Thus, the assumption of a constant vertical momentum transfer coefficient, N_z, is in doubt, and would suggest that if the coefficient is not a function of z, then perhaps it is a function of the total depth, d. It would then be a constant throughout the water column for any given depth.

Looking at Equation (4.28), the vertical velocity profile is described by,

$$u(z) \alpha \frac{z}{N_z} \tag{4.32}$$

As it would be reasonable to expect that the surface displacement of fluid would be proportional to the square of the wind speed rather than the flow depth, N_z is assumed to be a linear function of the depth in the form,

$$N_z = \bar{N}_z d \tag{4.33}$$

where

$\bar{N}_z$ = network constant defined by the above equation, (L/T).

Substituting in Equation (4.30), the model form of the wind induced vertical velocity profile is obtained,

$$u_k = \frac{1}{4} \bar{A}(k) K_w w_s^2 \cos\theta \ |\cos\theta| [d_2^2(\frac{d_2}{d} - 1) - d_1^2(\frac{d_1}{d} - 1)] / \bar{N}_z d(d_2 - d_1) \tag{4.34}$$

For a wind induced circulation model with tidal flow, Equations (4.34) and (4.31), superimposed on the tidal flow Equation (4.12), are used to describe the velocity field.

Figure 4.5 shows a typical measured profile in the 57 Acres canal network. Superimposed on the observed data are two plots which represent Equations (4.27) and (4.28), the vertical velocity distribution without and with the width correction factor. It can be seen that the introduction of the correction factor gives a good fit between the theoretical and observed profiles and serves to calibrate $\bar{N}_z$ and N_z used in the three-dimensional model.

One final problem remains, and that is the length of time a change in the wind requires to set-up the induced circulation discussed above. There appears to be little or no discussion on this in the literature, and the data gathered for this report is not long enough to identify the differences between fully developed conditions and the period of set-up. The theory described above is for a fully developed flow and can be calibrated directly only if the measured profile is also fully developed. The ultimate calibration is to run the model between successive periods of dye concentration sampling, and fit $\bar{N}_z$ to obtain the best fit.

To account for the period of set-up, a time constant can be read into the model over which the wind speeds and directions are averaged, and a resultant vector obtained. The parameter may also be used to calibrate the model, however it seems fairly clear that the value of this parameter must fall into a certain range. An upper limit to this value would reasonably be expected to be one-quarter of the tidal cycle as changes in the direction of flow for diurnal or semidiurnal tides would tend to break up the set-up.

4.4 Secondary Currents

The secondary current is an important phenomenon in mass transport because it usually offers the quickest means by which parcels of fluid can be transferred to the surface. Even though the velocities associated with this effect are small relative to velocities occurring along the channel, the length scales relative to the length of the channel are even smaller, and thus the time scale is also smaller.

This means that on an average, a parcel of fluid will reach the surface quicker due to secondary currents than due to upwelling in junctions, at dead-ends or at tidal entrances.

The importance of this action is that it transfers fluid from the benthic layer at the bottom of the channel to the surface where it may be reoxygenated, and then transfers it back down to the bottom as the secondary flow persists. This sometimes anoxic water is then "refreshed" and returned to the bottom where it can help cope with the oxygen demand.

The main cause of secondary currents is bends. These occur naturally in open watercourses and usually provide sufficient turnover to prevent stagnant zones of inactive water. However, in many of the man-made canal networks, the reaches are long and straight, particularly the dead-end reaches, and although secondary currents have been found to exist in straight channels, the magnitude of the associated radial component of the velocity is very much smaller than in bends. Thus, it has been observed that anoxic conditions frequently exist in the dead-ends of canal networks, and that when the waters are turned over by a strengthening or shifting wind, pungent odors are sometimes released, much to the chagrin of the residents.

In these straight canals, one method that has been suggested to improve water quality is to stagger roughness elements along each side of the channel to deflect the flow [Swenty, 1977; Parr and Christensen, 1977]. This has the effect of creating a series of bends, and of increasing the effective vertical diffusion coefficient, and the longitudinal dispersion coefficient. Of course, in this layout, the adjacent roughness element creates a helical flow in the opposite direction to the previous element and returns the fluid parcel to the bed. To overcome this, Christensen [1978] suggested an improvement would be to have a continuous bend in one direction causing a helical flow that would persist through the length of the canal and whose direction of rotation would only change with the tide. He theorized that an Archimedian spiral in the direction of the Coriolis effect would accomplish this.

A secondary current is produced when a water column is subjected to a transverse force in a bend. This force is called the centrifugal force, which is a reaction, by Newton's third law, to the centripetal force produced by the bend on the flow. Figure 4.6 shows diagramatically that when this force, which acts uniformly over the entire water column in a rectangular channel, is retarded at the bed by a friction force, a circulation is set up with the top layer moving towards the outside of the bend and the lower layer moving inward to conserve mass. When this circulatory pattern is superimposed on a longitudinal flow, a helical flow distribution is found (Figure 4.6).

In straight channels, the secondary currents are thought to be caused by the transport of momentum from the center to the sides, as Prandtl intuitively explained it [Ikeda and Kikkawa, 1976], but the mechanism is still not fully understood. The phenomenon has frequently been observed, but it was difficult to quantify because of the small scale of the flow. The usual method was to observe its effect on suspended sediments [Vanoni, 1946] or sand grains [Wolman and Brush, 1961]. With modern instrumentation such as the hot wire anemometer, more accurate measurements are possible, and tend to agree with Nikuradse's findings [1933] that the largest secondary currents are measured in the corners of the channel.

Recently several theoretical and numerical studies have been undertaken with varying degrees of success [Chiu and Lee, 1971; Ikeda and Kikkawa, 1976]. The latter authors compared their results for secondary flows in a corner with measurements obtained by Brundrett and Baines [1964], and found that they overestimated the magnitude of the velocities by 50 - 100 percent. For a wide channel, they found that

$$\frac{v}{u^*}, \frac{w}{u^*} > 0.2 \tag{4.35}$$

where

v = lateral velocity components (L/T)

w = vertical velocity component, (L/T).

Thus from Equation (4.5),

$$v, w > 0.024u \tag{4.36}$$

For Florida canals where tidal velocities on the order of 0.1 ft/sec are common, the lateral and vertical velocity components are at the threshold level of even the electromagnetic current meters, and smaller than any error term associated with lateral or vertical diffusion. Also, if the banks are sloping, the radial velocities are even smaller. Therefore, secondary currents in straight channels will not be considered, but rather the attention of this study will focus on the circulation due to bends.

The flow of water in bends is the subject of an extensive work by Rozovskii [1957]. He considered a logarithmic vertical velocity profile and derived the radical velocity component and an expression for the length over which this value decays after the curve ends. The question is whether such an analysis can be applied to a velocity field which includes flow reversals due to wind set-up or saline density flows.

Firstly, there appears to be no literature which attempts to derive a theoretical form for the secondary current in a bend under the influence of a velocity field with a flow reversal. Such a form, which would probably require a separate dissertation at least, is beyond the scope of this report, and as pointed out in the introduction, to model this effect using the full equations of continuity and momentum requires more of the model that the data collection methods can provide. So it is necessary to look to an intuitive argument that attempts to explain the physics of the secondary current process.

Looking at a flow reversal due to a wind set-up or salinity density current, in which there is no tidal action, there is zero net transport through any section. In the bend, secondary currents will then be initiated separately in the top and bottom layers but will act in the same rotational sense when viewed along the longitudinal x-direction. Because of the condition of zero net flow through any section, it is then argued that the two rotational flows will tend to cancel each other out as the momentum associated with each layer (Equation 4.28) is the same. Therefore, as it was assumed that as the closed form solutions for the velocity field could be superimposed, it follows that the same can be done for the secondary current effect. The only remaining nonzero net effect is then the tidal fluctuation which has a logarithmic vertical velocity profile, consistent with Rozovskii's basis of analysis.

Rozovskii [1957, ch. 2] considered the linear shear stress distribution that results in a logarithmic vertical velocity profile in turbulent flow in a wide channel, and developed an equation for the radial component of velocity,

$$v = \frac{1}{\kappa^2} u \frac{d}{r} [F_1(\eta) - \frac{\sqrt{g}}{\kappa C} F_4(\eta)] \tag{4.37}$$

where

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

r = radius of bend, (L)

η = z/d, dimensionless depth

C = Chezy's coefficient

and

$$F_1(\eta) = \int 2\frac{\ln\eta}{\eta-1} d\eta \tag{4.38}$$

$$F_4(\eta) = \int \frac{\ln^2\eta}{\eta-1} d\eta + 0.8(1 + \ln\eta) \tag{4.39}$$

This is the form of the radial velocity for a rough bed, and differs from the form which Fischer [1969] quoted incorrectly in the second term of Equation (4.39). Without this term, Equation (4.37), with $F_2(\eta)$ now instead of $F_4(\eta)$, is the form for the radial velocity with a smooth bed.

Using Chezy's equation in the form,

$$u = C(RS_f)^{1/2} \tag{4.40}$$

then from Manning's Equation (4.3),

$$C = 8.25 \sqrt{g}(\frac{R}{K})^{1/6} \tag{4.41}$$

Again as $R \simeq k$ and the ratio is raised to the 1/6 power, Chezy's coefficient can be written,

$$C = 8.25 \sqrt{g} \tag{4.42}$$

Thus, Rozovskii's expression for the radial velocity component is written,

$$v = 6.25\, u \frac{d}{r} [F_1(\eta) - F_4(\eta)/3.3] \tag{4.43}$$

Equations (4.38) and (4.39) cannot be solved as indefinite integrals because of the singularities at $\eta = 1$. Rather they may be solved using a numerical integration scheme tending towards the singularity. The results of this integration are given in Rozovskii [1957, p. 42]. Consider the integrand of Equation (4.38),

$$\frac{\ln\eta}{\eta-1} = \frac{\ln(\eta - 1 + 1)}{\eta - 1} = \frac{\ln(P + 1)}{P} \tag{4.44}$$

defining

$$p = \eta - 1$$

Using an infinite series expansion for the natural logarithm,

$$\frac{\ln(P + 1)}{P} = \sum_{i=0}^{\infty} \left(\frac{-P}{i + 1}\right)^{i} \tag{4.45}$$

which is valid for $0 < P \leq 1$.

Integrating,

$$\int\frac{2 \ln\eta}{\eta - 1}\, d\eta = \int\frac{2 \ln(P + 1)}{P}\, dp = 2 \sum_{i=1}^{\infty} (-1)^{i+1} \frac{P^{i}}{i^{2}} + a_0 \tag{4.46}$$

where

a_0 = constant of integration.

Matching with the curve in Rozovskii [1957, p. 42], $a_0 = 1.29$, and thus,

$$F_1(\eta) = 2 \sum_{i=1}^{\infty} (-1)^{i+1} \frac{P^{i}}{i^{2}} + 1.29 \tag{4.47}$$

As it stands, Equation (4.47) requires about thirty terms to be accurate to two decimal places. However, for a layered model, $F_1(\eta)$ is integrated over each layer to produce an average value, $\overline{F}_1(k)$ given by,

$$\bar{F}_1(k) = [2 \sum_{i=2}^{\infty} \frac{(-p)^i}{i^2(i+1)} + 1.29p]_{\eta_1}^{\eta_2}/(\eta_2 - \eta_1) \tag{4.48}$$

where

η_1, η_2 = lower and upper dimensionless depths of layer.

This form requires only about five terms to achieve the same degree of accuracy.

$F_4(\eta)$, given by Equation (4.39), is much more difficult to reduce to a similar form because integrations are required for terms like $p^i \ln/p$, where i is an integer. Thus, it was decided to fit a parabola to its shape given by Rozovskii [1957, p. 42] in the form,

$$\bar{F}_4(\eta) = -2.88 + 9.36\eta - 5.19\eta^2 \tag{4.49}$$

This form was based on the given values of $F_4(\eta)$ at η = 0.1, 0.5 and 1 so that when layer averaging was performed, errors would tend to cancel out to some degree. Thus, the layer average form of Equation (4.49) is,

$$\bar{F}_4(k) = [-2.88\eta + 4.68\eta^2 - 1.73\eta^3]_{\eta_1}^{\eta_2}/(\eta_2 - \eta_1) \tag{4.50}$$

The form of the radial velocity given in Equation (4.43) applies in the length of the curved portion of the bend only. Downstream this value decays to almost zero in a distance, L_d, given by Rozovskii [1975, p. 111] as,

$$L_d = \frac{C^2 d}{2g} \ln \frac{\kappa}{p} \tag{4.51}$$

where

p = permissible deviation from the background velocity.

For this model, p was chosen to be 0.02, and writing Chezy's C in terms of Nikuradse's equivalent sand roughness, k, Equation (4.51) becomes, using Equations (4.4) and (4.5),

$$L_d = \frac{8.25^2 d}{2} \ln(0.4/0.02) = 102d \tag{4.52}$$

Over the decay distance, L_d, the radial velocity decays exponentially to within 100p percent of 0 from its maximum value. $p = 0.02$ was chosen because the decay is then the same type of decay used to describe the flood tide boundary condition for the one-dimension model (Section 3.1.5). With this form, the radial velocity at any point of distance x away from the curved portion of the bend, and less than the decay distance, L_d, is given by,

$$v(x) = v \exp(-3x/L_d) \tag{4.53}$$

This velocity can then be averaged over the length of cell in which it acts to produce the model form of the lateral velocities due to secondary currents.

Good agreement was found between theoretical and observed radial velocities as seen in Figure 4.7. Here the velocities were measured just downstream of the crown of the bend in the south loop of the 57 Acres system (Figure 2.1), and were compared with those obtained theoretically from Equation (4.37). In each case, velocities on the order of 0.05 ft/sec were found, which is sufficient to cause a considerable amount of overturning of the water.

4.5 Density Induced Currents

In Section 4.4, the equations were developed for the velocity field under the influence of the wind shear stress and tide alone. This development of the hydrodynamics is valid for situations in which there is vertical homogeneity of nonpassive substances, such as saltwater. This situation is likely to be found in a canal network linked directly to the Intracoastal Waterway or ocean, or above the region of influence of the saltwater in a river estuary. However, in the transition region, the hydrodynamics will be affected by saline wedges intruding and retreating in the canal networks. McKeehan [1975] considered this to be the most significant effect in understanding the hydrodynamics of this type of canal network.

Most numerical models of stratified flow in fact only model the salt through a mass-transport equation, neglecting the effect of the density gradients on the continuity and momentum equations [King, Norton and Orlob, 1973; Hess and White, 1974]. This is a fair approximation if the flow is continuously stratified, but cannot be applied to the situation of a salt wedge in a canal because of the discontinuity in the density at the interface between the fresh and saltwaters. There has been a good deal of work done in trying to analyze the movement of a wedge, but mostly this is based on empirical formulae from laboratory and field measurements [Keulegan, 1958], or using grossly simplified equations to model the front of the wedge. Very little work has been done in numerical modeling the governing equations incorporating the density terms, except by people like Leendertse and Liu [1975]. However, this type of model is firstly very expensive to run, and secondly, its accuracy in tidal canals is doubtful because of the nature of the unknowns, such as bank and bed roughness, being modeled.

The mechanics of a saltwater wedge are depicted in Figure 4.8 and an example from the data collected on the Loxahatchee River shown in Figure 4.9. As the tide floods, a predominantly linear wedge intrudes into the canal. The forcing function for this wedge is the rising saltwater surface in the receiving waterbody. As the tide ebbs, the saltwater surface in the receiving waterbody falls, and the saltwater wedge in the canal receeds as the wedge loses the potential energy it gained during the flood tide.

The mechanism is very similar to that of a flood wave going overbank onto a flood plain [Walton and Price, 1975]. As the water rises overbank, the flow onto the flood plain is controlled by the dynamic wave that is the flood wave. As the water level in the main channel drops, the velocities associated with flow off the flood plain are then governed by bottom slope and friction. This is called the kinematic wave [Lighthill and Whitham, 1955].

The mechanism becomes even more complicated when a wind shear is applied at the water surface. The difficulties in deriving an accurate theoretical foundation for these complex interactions, dictate that an empirical derivation be investigated to match the conditions observed in the field.

With this in mind, the induced flow in a dead-end canal of a saltwater wedge in a tideless sea was considered. Under these conditions the net mass flux through any cross-section is zero. Studying the form of the saltwater wedge shown in Figure 4.9, the following assumptions were made (and shown in Figure 4.8),

1. the vertical velocity profile, $u_s(z)$, in the wedge is parabolic,

$$u_s(z) = u_1 z^2 \qquad \text{for} \qquad 0 \leq z \leq d_s \tag{4.54}$$

where

u_1 = constant depending on flow conditions, (1/LT)

d_s = depth of wedge, (L)

2. the induced counter-flow in the freshwater layer, $u_f(z)$, above the interface is constant,

$$u_f(z) = u_2 \qquad \text{for} \qquad d_s < z \leq d \tag{4.55}$$

where

u_2 = constant, (L/T).

The parabolic vertical profile was chosen firstly, because it seems to be in fairly good agreement with measured data. The second reason is that there is only the one calibration coefficient, u_1, (as u_2 can be found from conservation of mass). If a logarithmic form were chosen, which is certainly more expected, the model would have to be calibrated with not only a coefficient of proportionality, but also with a Nikuradse's equivalent sand roughness, k, which would depend on flow conditions within the wedge. Moreover, the parabolic profile is a reasonable approximation to the logarithmic profile, particularly when it is remembered that integrations are taking place to average conditions over vertical layers.

Keulegan [1958] showed that the velocity of the front of an advancing salt wedge in a tideless sea was a function of several parameters expressed as,

$$\frac{u_F}{u_\Delta} = f\left(\frac{L_w}{d}, \frac{u_\Delta d}{\nu}, \frac{d}{B}\right) \tag{4.56}$$

where

L_w = length of advancing front, (L)

ν = kinematic visocity, (L^2/T)

f( ) = function of ( )

and

u_Δ is the densimetric velocity expressed by,

$$u_\Delta = \left(\frac{\Delta\rho}{\rho} gd\right)^{1/2} \tag{4.57}$$

In a series of experiments measuring the frontal velocity of the wedge for the case in which two immiscible liquids were initially separated by a vertical barrier which was removed at time t = 0, he derived several formulae for the various values of the ratio d/B. For d/B = 2, he proposed,

$$\frac{u_F}{u_\Delta} = \left(1.75 + 0.16\left(\frac{u_\Delta d}{\nu}\right)^{-1/4} \frac{L_w}{d}\right)^{-1} \tag{4.58}$$

An examination of this formula however, suggests that this may not be a good equation to use to describe the movement of a salt wedge in these prototype conditions. The reason for this is that u_F, expressed in this manner, can vary dramatically with changes in L_w and d. In dead-end canals, the salt wedge may impinge on the dead-end thus altering the character of the flow. Also if a wedge is entering the canal during a flood tide and there is already the remnant of a salt dome left from the previous tidal cycle, then d becomes difficult to define as the velocity field acts throughout the column of salt water.

A better form here, might be a dependence on the slope of the interface. If a fluid in a closed frictionless basin is elevated at one end forming a sloping surface, and then released, a simple harmonic motion is set up as the rate of fluctuation of the surface is faster than the time the fluid takes to reach terminal velocity for a given acceleration. In a damped system, such as the movement of a saltwater wedge in a freshwater system over a rough bed, terminal velocities are reached much more quickly than the rate of response of the interface. Thus the velocity of the fluid is controlled by the component of the gravity term in the horizontal direction, which is proportional to the size of the angle the interface makes with the horizontal. Thus, the mean velocity, u_{sm}, at any vertical section in the wedge is given by,

$$u_{sm} = u_3 \, u_\Delta \sin \phi \tag{4.59}$$

where

u_s = constant, (L/T)

ϕ = angle between interface and positive x-direction, (degrees).

From Equation (4.54), the mean velocity at any vertical section in the wedge may also be written,

$$u_{sm} = \frac{1}{d_s} \int_0^{d_s} u_1 z^2 \, dz \tag{4.60}$$

Integrating and substituting Equation (4.59) into the resulting expression gives,

$$u_1 = 3 \, u_3 \, u_\Delta \sin \phi / d_s^2 \tag{4.61}$$

Thus, the vertical velocity profile can be written,

$$u_s(z) = 3 \, u_3 \left(\frac{\Delta\rho}{\rho} \, g d_s\right)^{1/2} \sin \phi \left(\frac{z}{d_s}\right)^2 \quad \text{for } 0 \leq z \leq d_s \tag{4.62}$$

Finally, the counter-flow velocity in the freshwater layer above the wedge, u_f, can be written from Equation (4.59), to conserve mass, as,

$$u_f = -u_3 \left(\frac{\Delta\rho}{\rho} \, g d_s\right)^{1/2} \sin \phi \, / (d/d_s - 1) \tag{4.63}$$

The expressions developed in Equations (4.62) and (4.63), are continuous forms of the vertical velocity profile for a channel of uniform width. To extend these expressions to the three-dimensional model, an averaging process and a width correction factor are used in exactly the same way as described for the wind induced circulation case in Section 4.3.

Unfortunately, on programming, the method was found to be very unstable. This was due to the variability of the $\sin\phi$ term in Equations (4.62) and (4.63). If a sharp discontinuity appears in the longitudinal profile, the velocity term associated with the flow between adjacent cells can become very large and draw most of the fluid from one of the cells. At the next time step fluid from adjacent cells on either side flows in and the discontinuity becomes ever bigger than before. Furthermore, attempts to damp out the instability merely defined the damping term as the condition determing the profile, thus overriding the governing equation.

It was felt that the problem was due to the specification of the angle ϕ locally. In fact, it should more realistically be the slope of the saltwater surface out in the receiving water, particularly during the flood tide, and thus the densimetric velocity, u_Δ , would have to be a function of the depth of the receiving water. Clearly, this is not an easy thing to measure, and the ebb tide expression remains unclear. Thus, an alternative expression was sought.

A much simpler condition, and probably a more accurate expression for the flood tide case, would be a mean velocity at the tidal entrance, $u_{sm_{TE}}$, proportional to the rate of rise of the interface at that point in a similar manner to a dynamic flood wave going overbank in a river [Walton and Price, 1975],

$$u_{sm_{TE}} = u_4 \frac{dd_s}{dt} \tag{4.64}$$

where u_4 = constant.

If the salt wedge is assumed to take its simplest form, a triangular distribution, a theory can be developed to predict its passage through the reaches of a canal network. Consider a single rectangular, prismatic canal (the model can easily be extended to a trapezoidal cross-section, but is not done so here to simplify the equations), then during the flood tide, the length of the wedge, L_w, before it impinges on a dead-end is given by,

$$L_w = 2\, u_4 d_{s_{TE}} \tag{4.65}$$

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

$$d_s = d_{s_{TE}} \left(1 - \frac{x'}{2u_4 d_{s_{TE}}}\right) \tag{4.66}$$

where

x' = distance from tidal entrance, (L).

From the linearity of the governing equation then, the mean velocity in the wedge, u_{sm}, at x' is,

$$u_{sm} = u_4 \frac{dd_s}{dt} \tag{4.67}$$

Now from Equation (4.60),

$$u_1 = 3\, u_{sm}/d_s^2 \tag{4.68}$$

thus from Equations (4.54), (4.67) and (4.68), the vertical velocity profile in the wedge is given by,

$$u_s(z) = 3\, u_4 \frac{dd_s}{dt} \left(\frac{z}{d_s}\right)^2 \tag{4.69}$$

In a similar manner to Equation (4.63), the velocity in the freshwater layer is written,

$$u_f = - u_4 \frac{dd_s}{dt} / (\frac{d}{d_s} - 1) \tag{4.70}$$

Once the wedge impinges on the dead-end of the canal, the elevation of the wedge above its low tide value, $d_{s_{DE}}$, is given by,

$$d_{s_{DE}} = d_{s_{TE}} (\frac{2u_4 d_{s_{TE}}}{L} - 1) \tag{4.71}$$

where

L = length of canal, (L).

Then, at any point of distance x' from the tidal entrance, the elevation of the salt wedge above its low tide value is written,

$$d_s = d_{s_{TE}} (1 + \frac{2x'}{L} (u_4 d_{s_{TE}} - 1)) \tag{4.72}$$

and the mean velocity through this section is,

$$u_{sm} = u_4 \frac{dd_{s_{TE}}}{dt} (1 - \frac{x'}{L}) \tag{4.73}$$

As before, from Equations (4.54), (4.68) and (4.73), the vertical velocity profile in the saltwater layer at this section is,

$$u_s = 3u_4 \frac{dd_{s_{TE}}}{dt} (1 - \frac{x'}{L}) (\frac{z}{d_s})^2 \tag{4.74}$$

and the constant velocity in the freshwater layer can be written,

$$u_f = - u_4 \frac{dd_{s_{TE}}}{dt} (1 - \frac{x'}{L}) / (\frac{d}{d_s} - 1) \tag{4.75}$$

For the ebb tide, the analogy with the flood wave returning inbank as the flood wave receeds, may be followed. Here, the velocity of the kinematic wave is proportional to the depth and thus the front of the wedge remains fixed. The velocities are then induced by the decrease in the interface elevation in response to the rate of fall at the tidal entrance. Assuming to first order, that the interface in the canal remains linear, then the depth at any point of a wedge that does not impinge on the dead-end is,

$$d_s = d_{s_{TE}}\left(1 - \frac{x'}{2u_4 d_{s_{HT}}}\right) \tag{4.76}$$

where

$d_{s_{HT}}$ = elevation of interface of salt layer, above its low tide value, at high tide (L)

and the mean velocity,

$$u_{sm} = \frac{1}{2}\frac{dd_{s_{TE}}}{dt}\left(\frac{L_w - x'}{L_w d_s}\right)^2 \tag{4.77}$$

From Equations (4.54), (4.68) and (4.77), the vertical velocity profile in the saltwater layer is,

$$u_s(z) = \frac{3}{2}\frac{dd_{s_{TE}}}{dt}\left(\frac{L_w - x'}{L_w d_s}\right)^2\left(\frac{z}{d_s}\right)^2 \tag{4.78}$$

and the freshwater velocity is,

$$u_f = -\frac{1}{2}\frac{dd_{s_{TE}}}{dt}\left(\frac{L_w - x'}{L_w d_s}\right)^2 / \left(\frac{d}{d_s} - 1\right) \tag{4.79}$$

Finally for the case in which the wedge does impinge on the dead-end, the elevation of the interface above its low tide value and any point x' is given by,

$$d_s = d_{s_{TE}}\left(1 - \frac{2x'}{L}\left(1 - \frac{u_4 d_{s_{TE}}}{L}\right)\right) \tag{4.80}$$

and the mean velocity is,

$$u_{sm} = \frac{dd_{s_{TE}}}{dt} \left(\frac{L - x'}{d_s}\right)\left(1 - \left(1 + \frac{x'}{L}\right)\left(1 - \frac{u_4 d_{s_{TE}}}{L}\right)\right) \tag{4.81}$$

From Equations (4.54), (4.68) and (4.81), the vertical velocity profile in the saltwater layer is,

$$u_s(z) = 3 \frac{dd_{s_{TE}}}{dt} \left(\frac{L - x'}{d_s}\right)\left(1 - \left(1 + \frac{x'}{L}\right)\left(1 - \frac{u_4 d_{s_{TE}}}{L}\right)\right)\left(\frac{z}{d_s}\right)^2 \tag{4.82}$$

and the constant velocity in the freshwater layer is written,

$$u_f = \frac{dd_{s_{TE}}}{dt} \left(\frac{L - x'}{d_s}\right)\left(1 - \left(1 + \frac{x'}{L}\right)\left(1 - \frac{u_4 d_{s_{TE}}}{L}\right)\right) \Big/ \left(\frac{d}{d_s} - 1\right) \tag{4.83}$$

This method was extened to the case of the canal network with trapezoidal cross-sections and programmed. Data from the Loxahatchee River was used to test the program, and as can be seen in Figure 4.10, fairly good agreement is found. It will also be seen in Chapter 8 that the transient dye concentration profile is fairly well followed, giving some confidence to the theory.

The Loxahatchee River canals are fairly typical canals on both the east and west coasts of Florida in their geometric features, particularly the length. Thus, it was not possible to study the variability of the coefficient u_4 used in Equation (4.64). This value was calibrated to fit the above test case and is assumed to be a reasonable value for similar canals. However, it is possible that the coefficient is dependent on the length of the canal, and thus if possible it should be calibrated on a similar canal to any design canal considered.

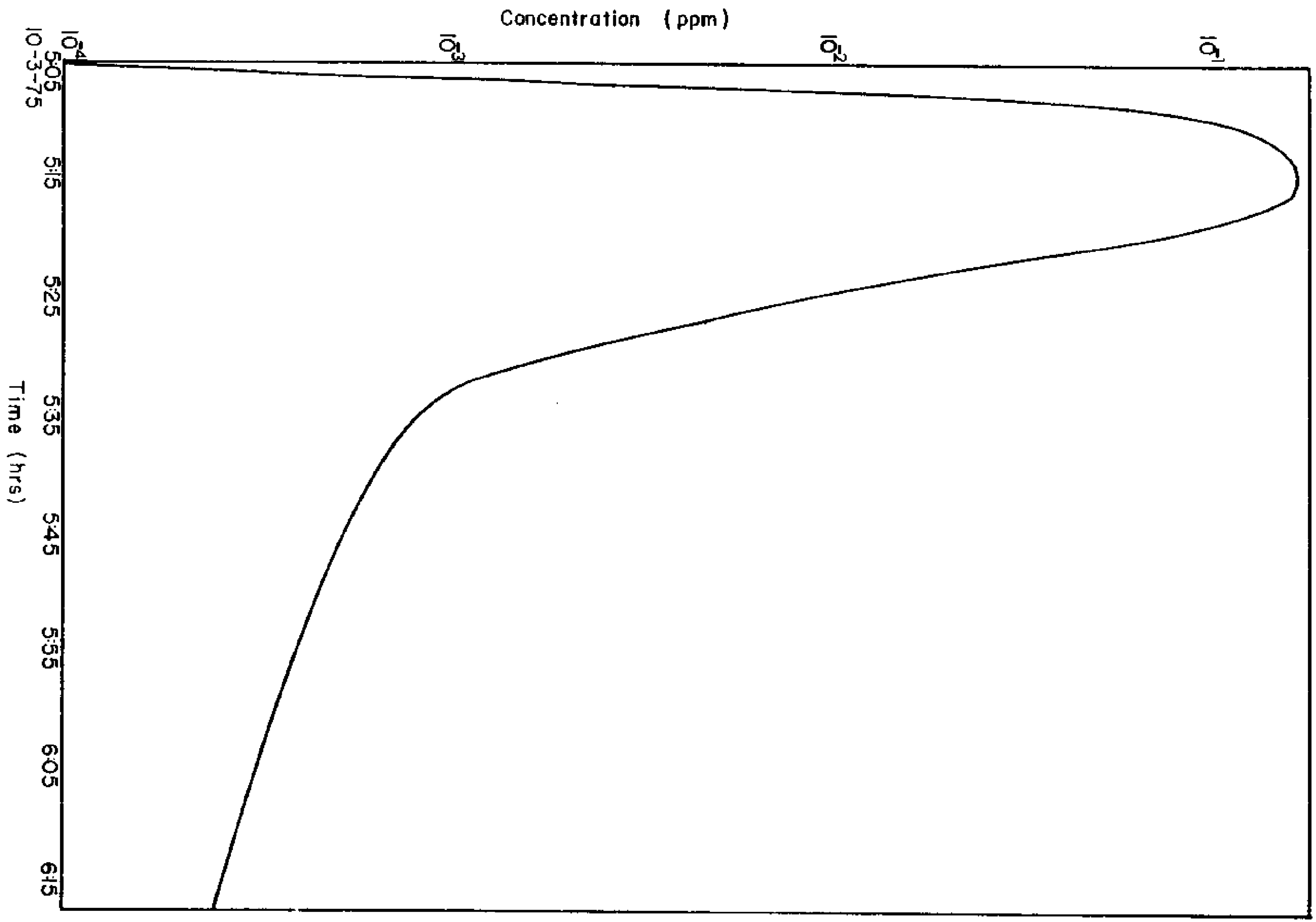


Figure 4.1 - Example of Typical Non-Gaussian Concentration Profile.

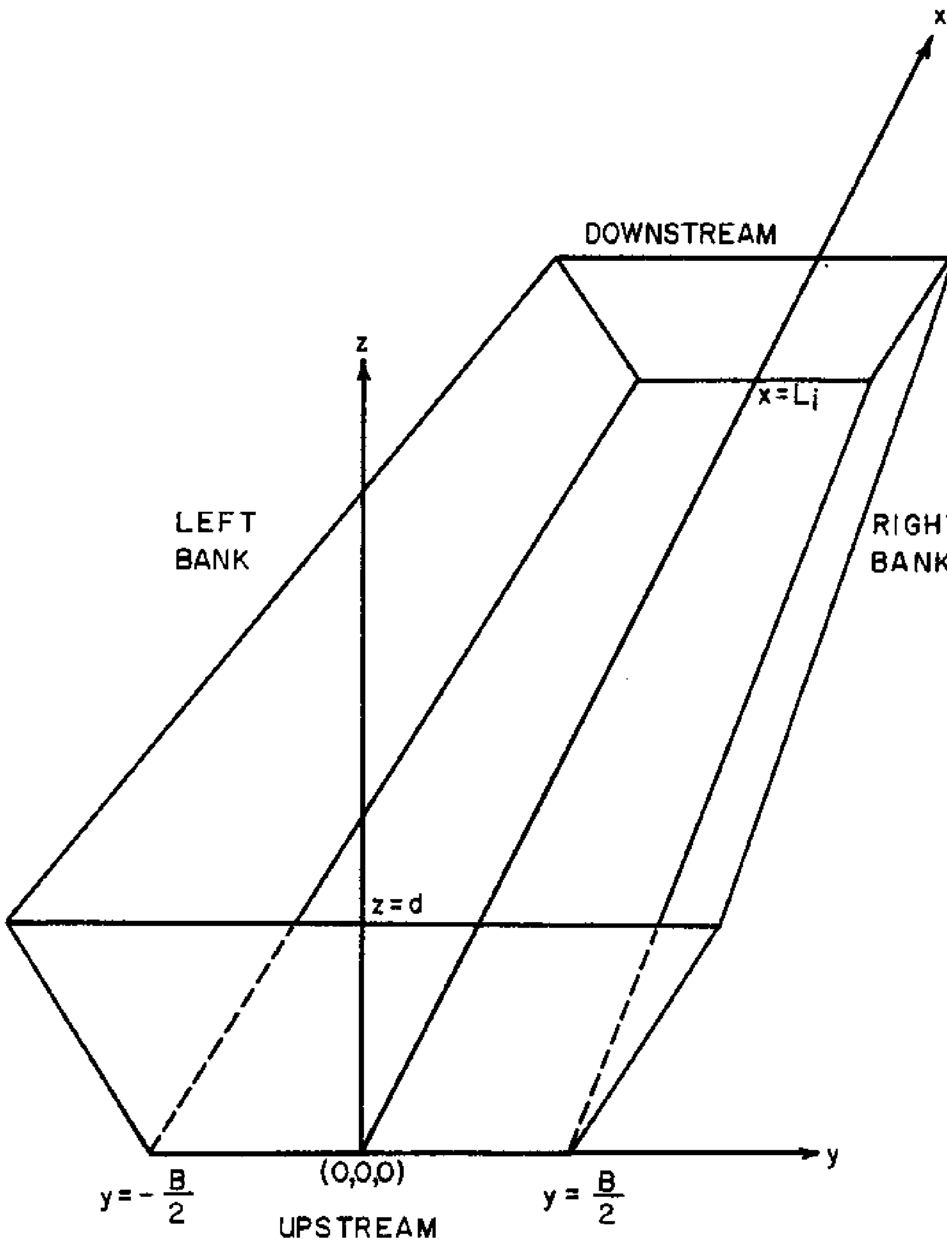


Figure 4.2 - Definition of Coordinate System.

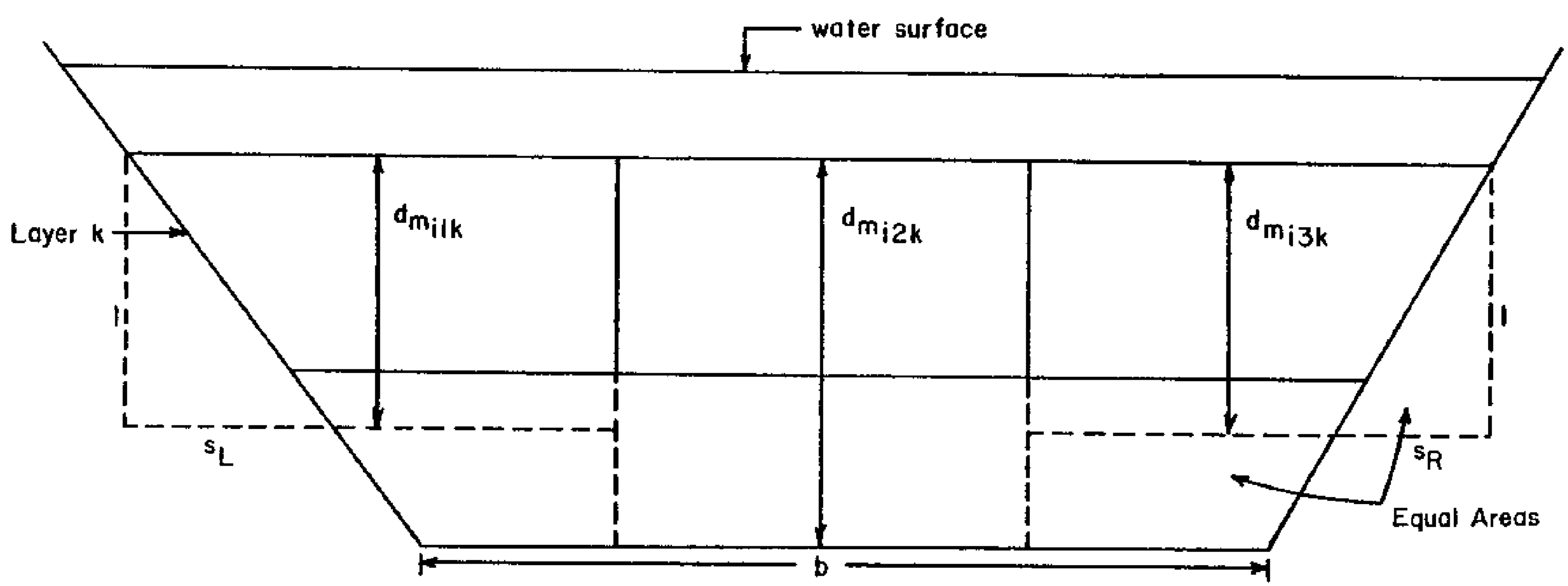


Figure 4.3 - Schematic Drawing of Mean Depths in Lateral Cells of a Vertical Layer.

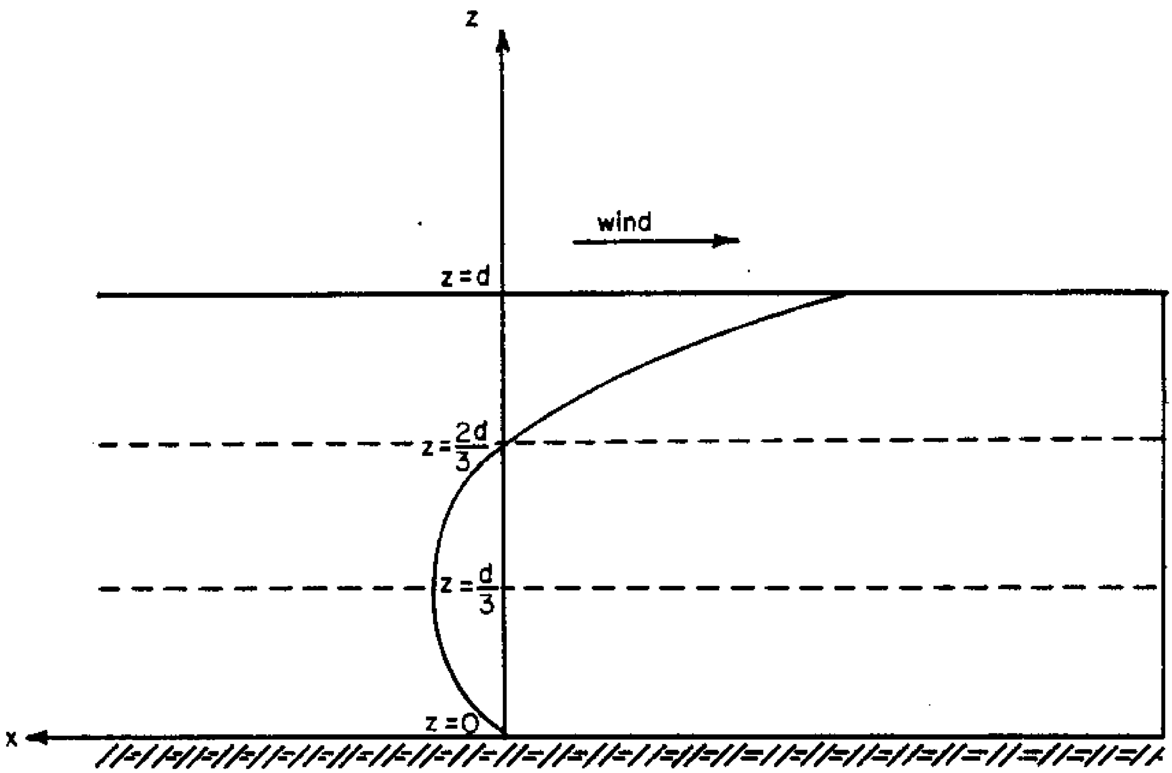


Figure 4.4 - Theoretical Wind-Induced Vertical Velocity Profile.

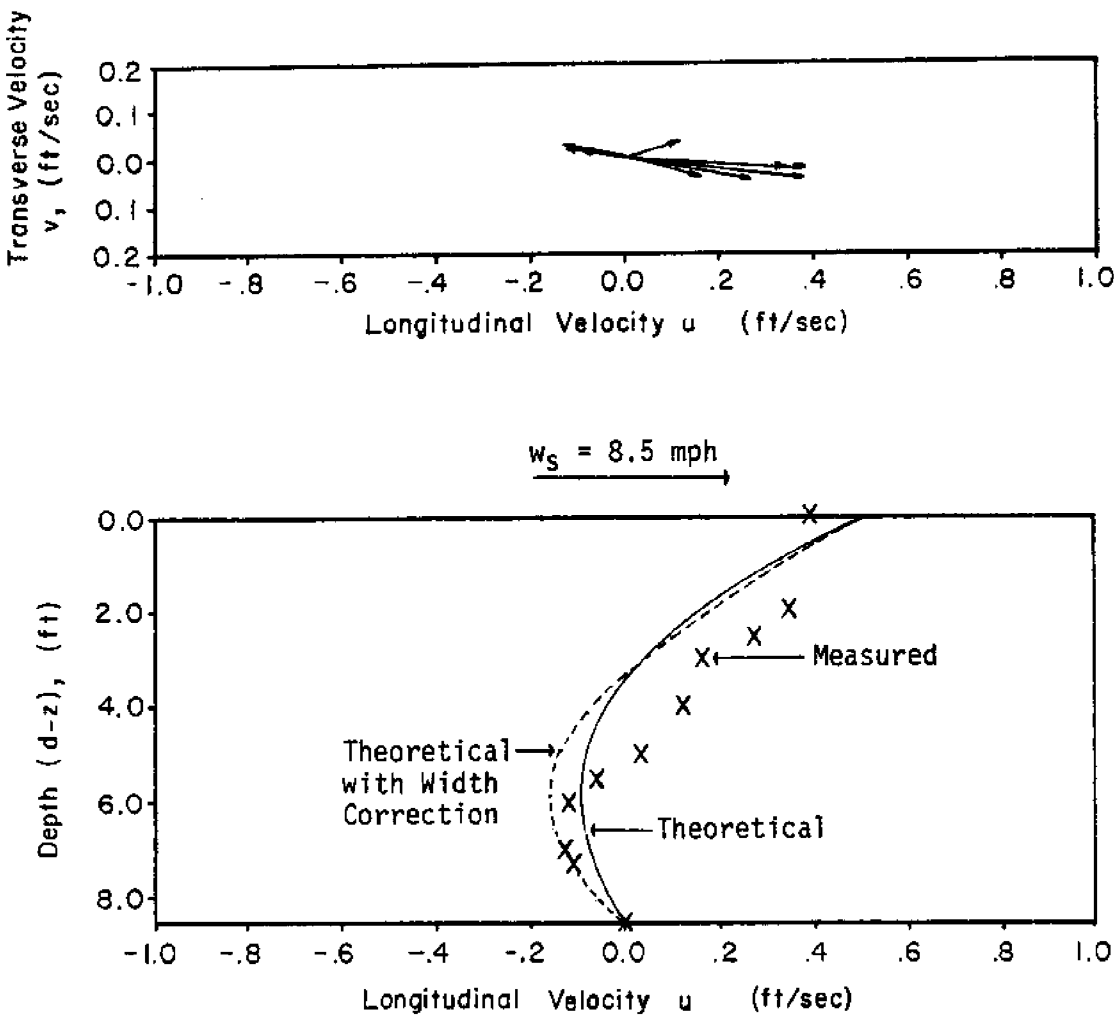


Figure 4.5 - Comparison Between Observed and Theoretical Wind-Induced Vertical Velocity Profiles, With and Without Width Correction (N_z = 0.002 ft^2/sec) - 57 Acres.

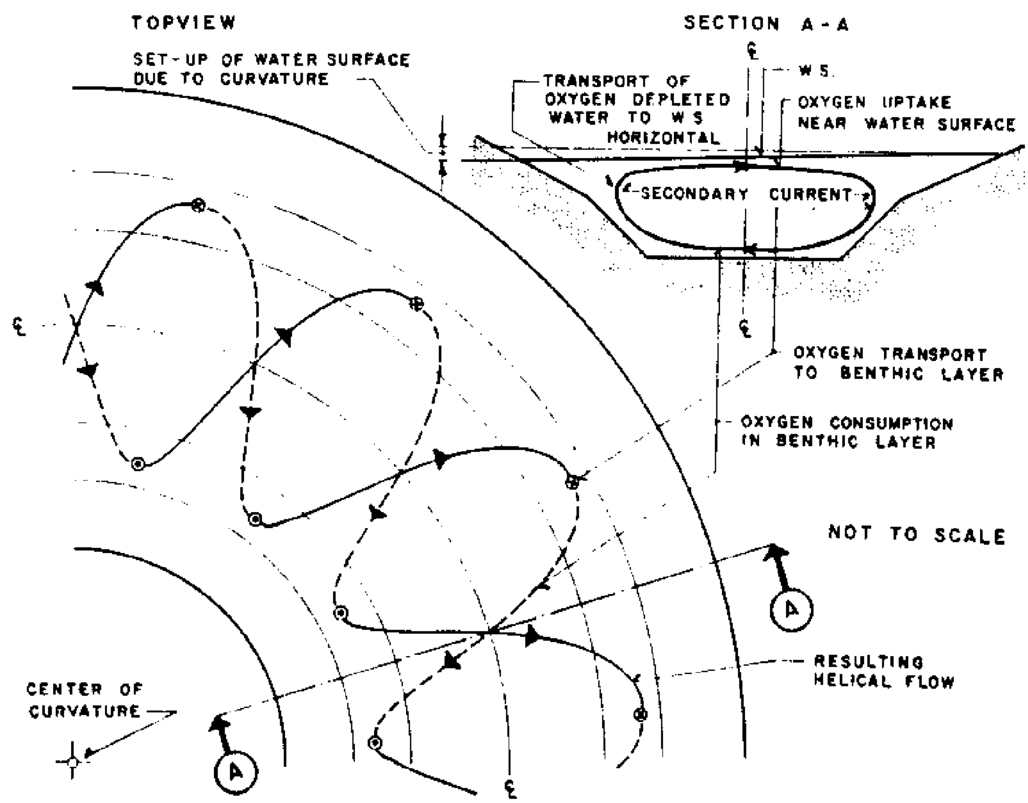


Figure 4.6 - Schematic Drawing of Helical Current Induced by a Bend.

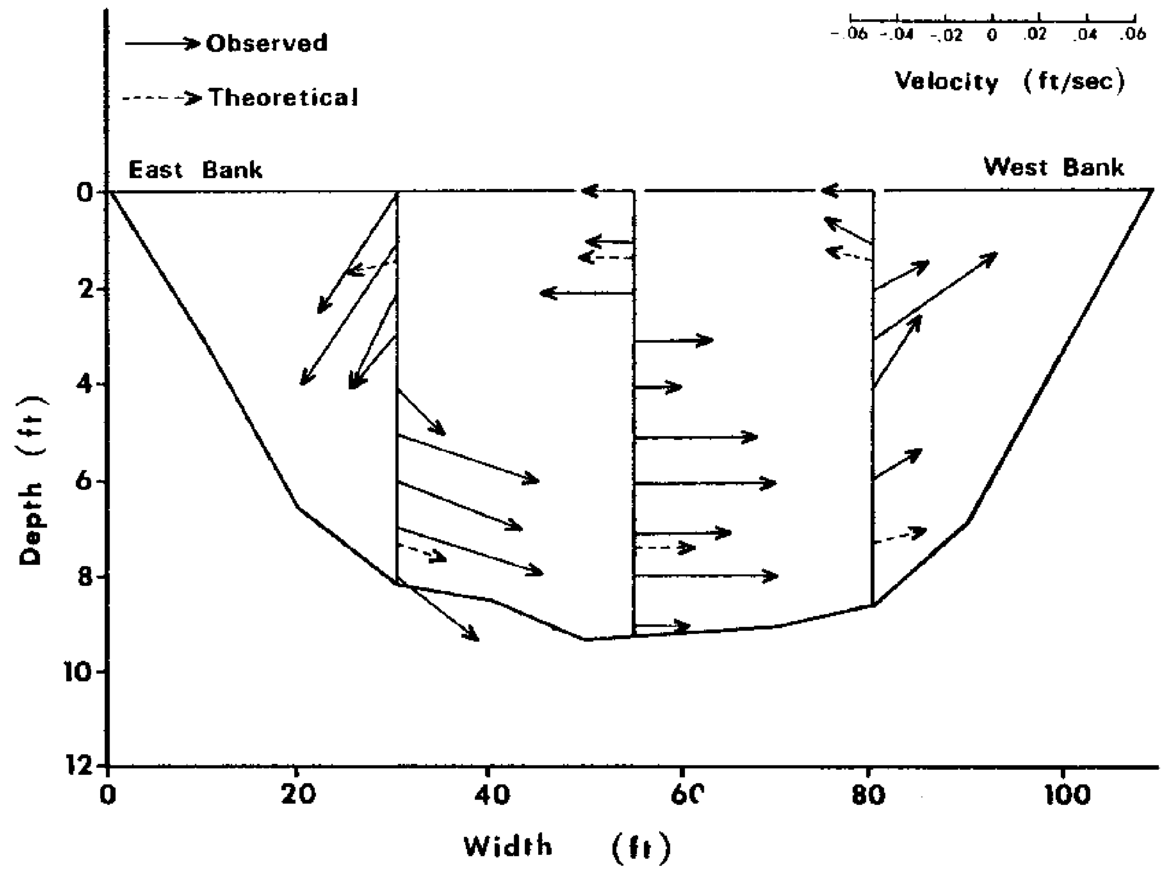


Figure 4.7 - Comparison Between Observed and Computed Lateral Velocities Induced by Bend in South Loop of 57 Acres System.

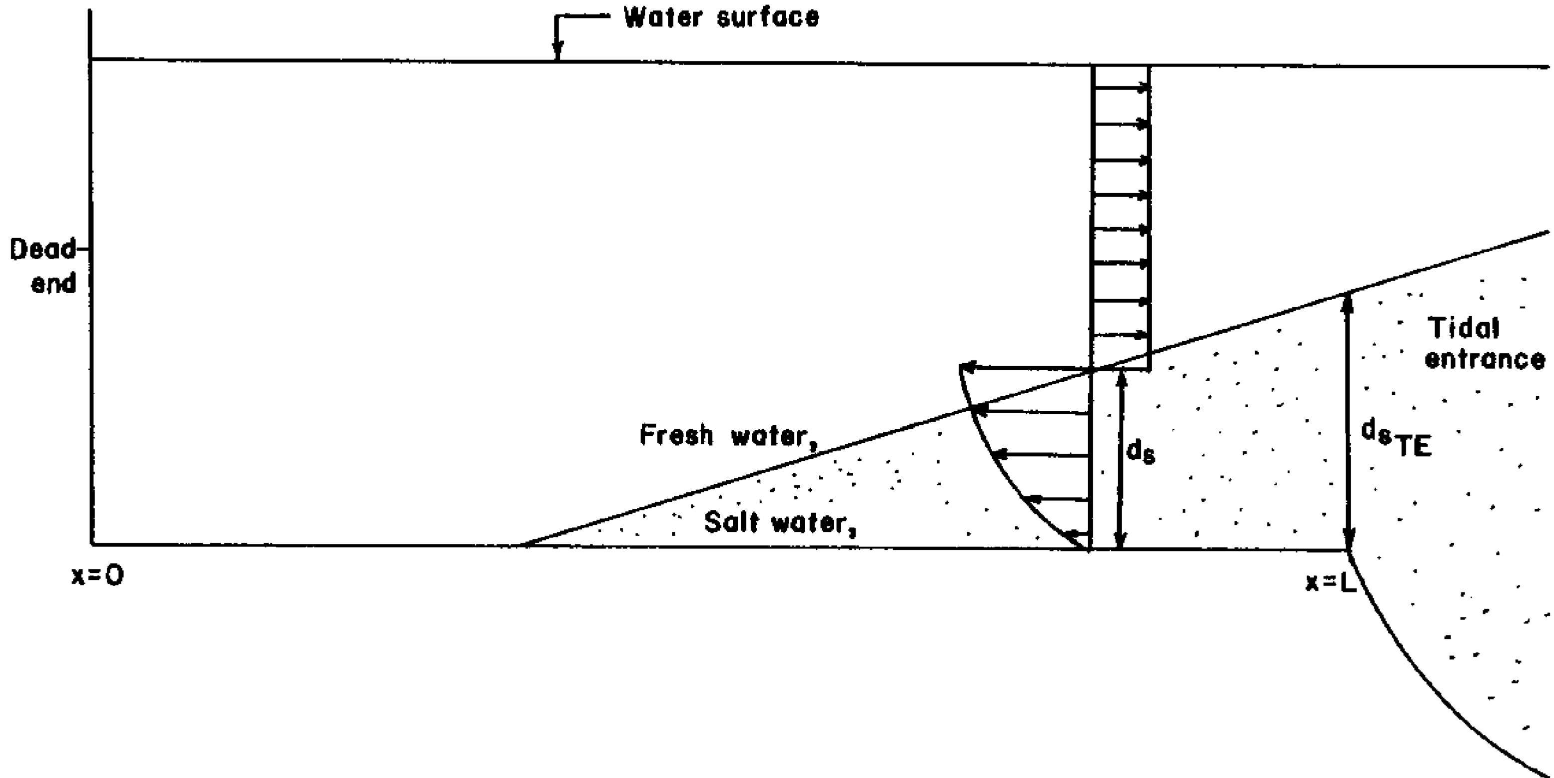


Figure 4.8 - Schematic Drawing of a Salt Wedge Entering a Canal Showing Definitions.

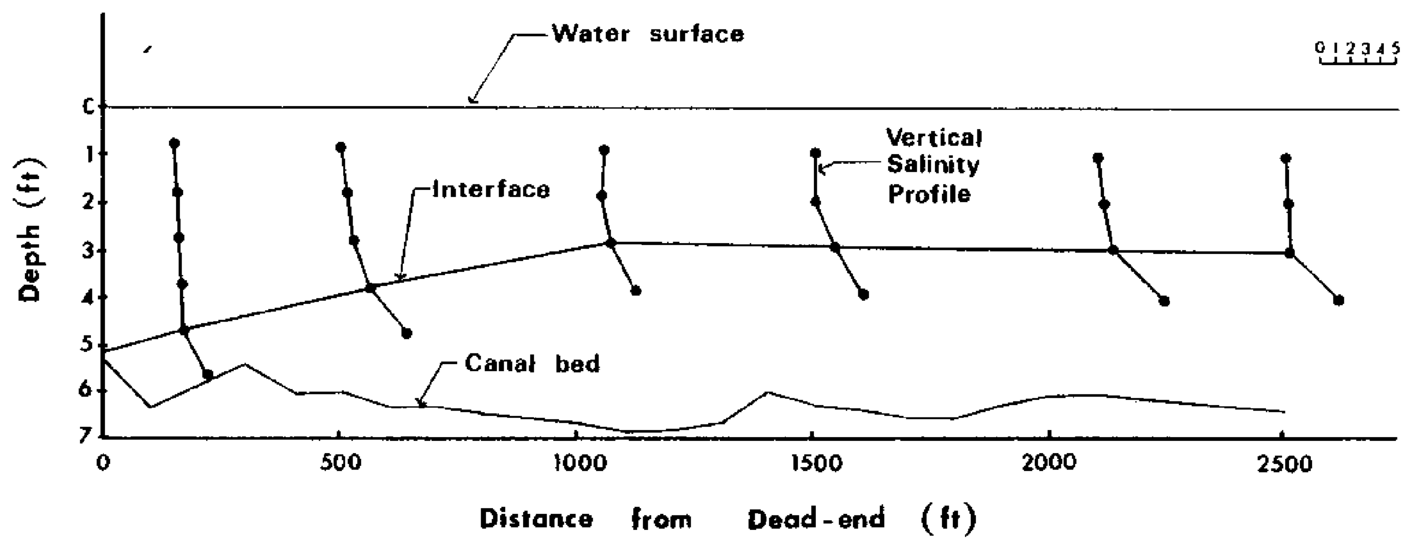


Figure 4.9 - Typical Salinity Profiles in the Loxahatchee River North Canal, Showing Presence of a Saltwater Wedge.

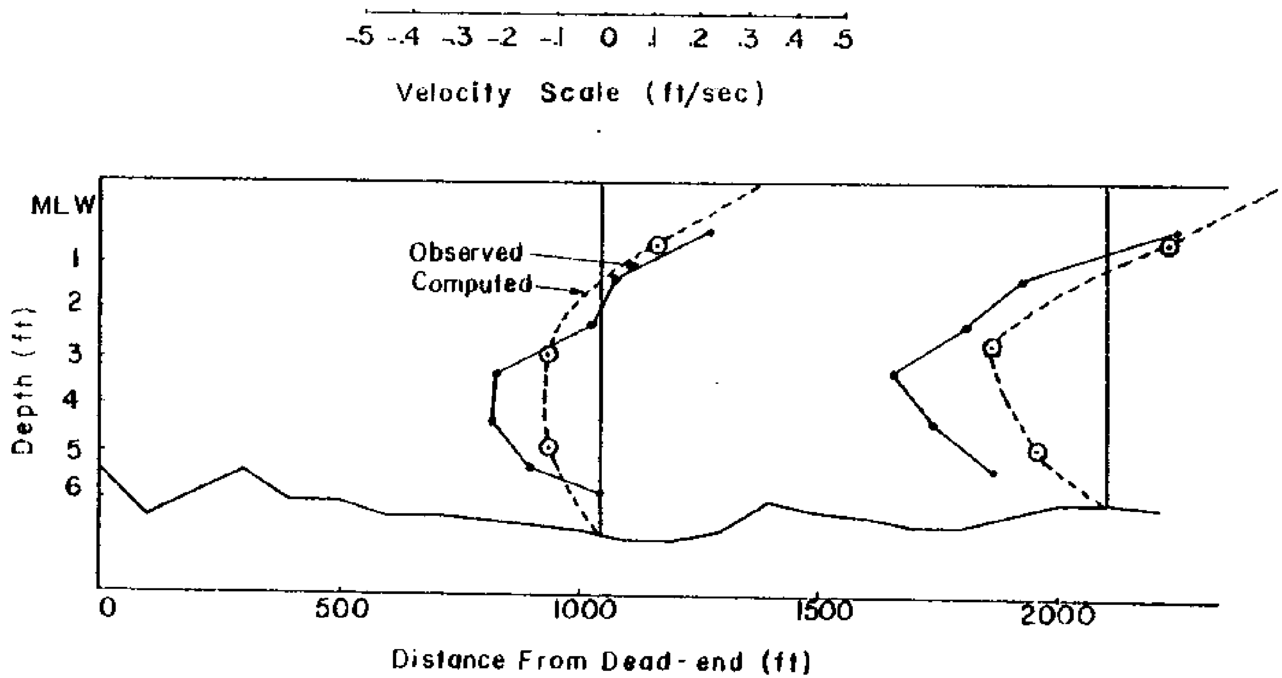


Figure 4.10 - Comparison of Observed and Computed Velocity Profiles for Loxahatchee River Site.

CHAPTER 5

THREE-DIMENSIONAL MASS-TRANSPORT EQUATION

5.1 Transport Mechanisms

The derivation of the three-dimensional mass-transport equation was briefly discussed en route to deriving a one-dimensional model form. The equation was written,

$$\frac{\partial c}{\partial t} + \frac{\partial}{\partial x}(cu) + \frac{\partial}{\partial y}(cv) + \frac{\partial}{\partial z}(cw) = \frac{\partial}{\partial x}\left(E_x \frac{\partial c}{\partial x}\right) + \frac{\partial}{\partial y}\left(E_y \frac{\partial c}{\partial y}\right) + \frac{\partial}{\partial z}\left(E_z \frac{\partial c}{\partial z}\right) + r_p \tag{5.1}$$

The derivation of this equation has been the topic of many works and will be considered without proof as the model transport equation. The discussion in this chapter rather centers upon the model forms of the three diffusion coefficients, E_x, E_y and E_z.

It is convenient at this time to introduce the distinction between a diffusion coefficient and a dispersion coefficient used in this report. A *diffusion* coefficient is defined as a coefficient representing 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 similarly defined as a coefficient representing a transport analogy which is averaged over a length scale associated with the transport mechanism. In the discussion on one-dimensional modeling, a longitudinal dispersion coefficient was obtained by averaging the three-dimensional mass-transport equation over the cross-sectional area. Similarly dispersion coefficients may be obtained by averaging over just one coordinate direction, or over the length scale of a computational cell. Thus diffusion coefficients are converted to dispersion coefficients by cell averaging, and their associated dimensionless coefficients obtained from field calibration.

The spreading of a substance, such as a dye cloud, involves two component parts which are caused by convection (or advection) and turbulent diffusion. By definition, convection is the transport of a substance with the velocity of the fluid, whereas advection is defined merely as transport in the direction of the flow. The further distinction is made with the meterological form of convection, which is vertical transport due to thermal gradients. In this report, it is assumed that all substances are transported with the velocity of the fluid and hence convection is being discussed.

The difference between convection and diffusion, or dispersion, is really the scale with which the phenomenon can be measured. If, for example, a substance such as a dye cloud, is instantaneously introduced into a fluid, then its initial transport will be governed by the small time scales associated with turbulent flow, and by spatial variability locally in the velocity field. As the substance begins to spread out under the influence of these small scale eddies, its transport becomes governed by eddies of increasing size up to the size of the largest eddy present in a tidal watercourse, the tide itself.

In the development of Equation (5.1), the diffusive terms on the left hand side of the equation are formed by considering what is called the turbulent fluctuation of the velocity from the turbulent time mean value, $\bar{u}$, say, in the longitudinal direction. Thus

$$u = \bar{u} + u' \tag{5.2}$$

where

u' = instantaneous velocity in x direction, (L/T).

Similarly, the fluctation, c', from the turbulent time mean value of the concentration, $\bar{c}$, is written

$$c = \bar{c} + c' \tag{5.3}$$

where

c = instantaneous concentration, (dimensionless).

Substitution of terms like these in the initial conservation of mass in an elemental volume [Harleman, 1966], yields terms like $\overline{u'c'}$ which are analogous to Reynolds' stress in turbulent flow.

The use of these terms is to simply say that not enough is known about the true forms of these variables, or that the time interval in the model is greater than the time scale of the turbulent eddies, and so a mechanism must be developed to distribute the substance spatially more than that predicted by following the mean velocity during the time interval.

This is usually done by considering an analogy with Fick's first law of transport due to molecular diffusion. The turbulent analogy is written in the x direction,

$$\overline{u'c'} = - E_x \frac{\partial c}{\partial x} \tag{5.4}$$

and is called the transport due to turbulent diffusion in the x-direction. Similar expressions are obtained for E_y and E_z, in the y and z-directions, respectively.

In a similar manner then, as described before, the dispersive transport $E_\ell \partial c/\partial x$ in a one-dimension model is defined, again using a Fickian analogy. This analogy compensates for the fact that the hydrodynamics are defined in one direction only, and that the spatial variations in the velocity field are providing a three-dimensional transport mechanism.

This idea can be extended to the discretization scheme of any numerical model. Analogous 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 established to include this effect. Commonly in three-dimensional models, the turbulent diffusion coefficient in a given direction will be described as a mean of coefficients at the centers of adjacent cells in that direction. This is alright if the variation in the gradient of the velocity field is constant or linear, but for higher order variations the fitting of straight line segments might cause information to be lost, particularly if a coarse grid is being used. To account for this subgrid scale variation, the diffusion coefficients can be adjusted by averaging over the computational cell, thus becoming dispersion coefficients.

Another use of the diffusion term is to account for unknowns in the velocity field. Such an unknown of importance in this study is the circulation at slack tide. Holley and Harleman [1965], were amongst the first to study the mechanism of dispersion in oscillatory flow, and developed a form of the longitudinal dispersion coefficient for oscillatory flow based on the work of Taylor [1954] and Elder [1959],

$$E_{\ell} = 20.2Ru^{*}(t) \tag{5.5}$$

Theoretically, assuming no external forcing functions such as wind or density gradients to exist, E_{ℓ} becomes zero at slack tide. This condition, of course, is unrealistic in tidal canals particularly near the dead-ends of the canals, because it suggests that there is practically no transport taking place at this time. However, it is well known that even during these periods, there is still considerable activity due to eddies whose mechanisms are not entirely understood, but which are probably caused by residual momentum effects or the introduction into the canal systems of oceanic eddies whose time scales are much smaller than the tidal period, but larger than the time scales associated with turbulent fluctuations. In fact, on several field trips, it was felt from some of the velocity readings, that eddies with periods on the order of three seconds were present, which is much larger than turbulent scales, and whose origins were not understood.

With this in mind, a more realistic form of Holley and Harleman's oscillatory dispersion coefficient might be

$$E_{\ell} = 20.2Ru^{*}(t) + E_c \tag{5.6}$$

where

E_c = background dispersion coefficient, (L^2/T).

In the ensuing sections on the various diffusion and dispersion coefficients, this idea will be incorporated into the model forms of these coefficients.

5.2 Longitudinal and Lateral Diffusion Coefficients

The choice of the model forms of the diffusion coefficients is a difficult one indeed as so many suggested forms have been presented based on "laws" or empirical data merely reflecting our ignorance as to the actual mechanisms operating. Most of these forms are based on assuming that the momentum transfer coefficients, N_x, N_y and N_z, and the turbulent diffusion coefficients, E_x, E_y and E_z, are interchangeable. Indeed, there is a foundation of evidence for this assumption based on the work of several investigators [Jobson and Sayre, 1970; Schiller and Sayre, 1973; Dlubac, 1976].

One of the most common of these laws is the so-called "4/3-power law" which is based on the assumption of isotopic turbulence [Richardson, 1926],

$$E_x = k\ell^{4/3} \tag{5.7}$$

where

k = constant of proportionality, $(L^{2/3}/T)$

ℓ = length scale, (L).

Isotropic turbulence does not exist in any naturally occurring fluid, but many investigators have gathered data to determine if such a law is approximately valid, or to determine a more realistic form of the power for their data.

One such example is from the work of Okubo [1971], who found that the best fit for his data was

$$E_x = 0.01\ell^{1.15} \tag{5.8}$$

From this data, and from the results of other investigators, it appears that the "4/3 power law" gives a good approximation for diffusion in the sea [Christodoulou et al, 1976], however, Zeidler [1976] has shown that this form of the diffusion coefficient is not acceptable near the shoreline due to bottom and side effects. In canal networks, bottom and side effects are the dominant mechanism affecting the form of the diffusion coefficients, and so the "4/3 power law" method was not used in this study.

Another drawback is the selection of a suitable length scale, ℓ. In canals and rivers this is usually taken to be some characteristic length of the geometry, such as the width, but this could lead to potential errors particularly in tidal canals, due to the presence of a number of eddy sizes ranging from the turbulent scale upward.

The fact that the bottom and side effects are such an important part of the diffusion mechanism, leads us to follow the work of Taylor [1953, 1954], Elder [1959] and Aris [1956] more closely. Their work was more directly affected by the forms of the boundary roughness.

Taylor [1954] showed that for a flow which obeyed the "universal" distribution of velocity in a pipe [Goldstein, 1938, p. 336],

$$\frac{u_m - u}{u^*} = f(z) \tag{5.9}$$

where

u_m = mean velocity, (L/T)

the longitudinal diffusion coefficient, E_x, could be written,

$$E_x = 0.104Ru^* \tag{5.10}$$

This form of the longitudinal diffusion coefficient should then hold near any solid boundary, and indeed is assumed to hold for any velocity field which exhibits logarithmic profiles. The problem arises when the flow is non-logarithmic as evidenced by flow reversals due to wind shear and density gradients. These phenomena occur mainly in the vertical direction and it is argued that, as Equation (5.10) holds near the solid boundaries, it also holds for any horizontal plane in the flow.

For circulation due to wind stress, the vertical velocity profile is only a function of z as shown in Chapter 4. For density flows due to salinity gradients, although lateral uniformity is assumed, the form of the interface is a function of both x and z. However, the interface is a discrete point at which one homogeneous fluid of density ρ is assumed to change to a second homogeneous fluid of density $\rho + \Delta\rho$. If it is assumed that an equation like Equation (5.10) holds for each fluid, the solutions can be matched across the interface. Thus the longitudinal diffusion coefficient, E_x, can be written,

$$E_x = K_x R u^* \tag{5.11}$$

Considering a wide channel in which,

$$R \approx d \tag{5.12}$$

and using Equation (4.5), the longitudinal diffusion coefficient can be written,

$$E_x = K_x du/8.25 \tag{5.13}$$

Traditionally, the lateral diffusion coefficient, E_y, is related to the longitudinal form by a constant of proportionality. For example, Elder [1959] wrote the lateral diffusion coefficient as,

$$E_y = 0.23 R u^* \tag{5.14}$$

in which he depth-averaged the coefficient and thus produced a dispersion coefficient. In spite of this fact, a number of people still use Equation (5.14) within a three-dimensional numerical analysis.

The lateral transport mechanism is the same as that discussed in Chapter 4 where the lateral variation in the velocity field was considered. Assuming a logarithmic velocity profile based on the mean depth in the section of flow considered, the mean velocity, v, from one cell to a laterally adjacent cell will be governed by the velocity difference between the cells (Figure 5.1).

$$v \propto u_j - u_{j-1} \tag{5.15}$$

where

u_j, u_{j-1} = velocities in laterally adjacent cells, (L/T).

Assuming logarithmic velocity profiles,

$$v \propto u^* \ln\left(\frac{d_{m_i}}{d_{m_{i-1}}}\right) \tag{5.16}$$

and hence

$$v \propto u^* \tag{5.17}$$

Now even though the vertical profiles are not logarithmic, it may be assumed that horizontal profiles are based on lateral logarithmic profiles from the channels' banks. Thus an expression similar to Equation (5.17) can be derived and the interdependence of the longitudinal and lateral velocity profiles shown.

A number of investigators as reported by Fischer [1969], have measured the depth-averaged lateral diffusion coefficient to vary quite considerably. For example, Yotsukura et al [1968] found the dimensionless diffusion coefficient to be 0.6 for a curved portion of the Missouri River. The fluctuation of the coefficient is undoubtedly due to the fact that secondary current effects were lumped into the diffusion coefficients and not into the convective terms. In this model, however, an expression for the velocities induced by secondary currents has been included in the model hydrodynamic equations, and so this coefficient should not vary as much as observed by other investigators who do not model this effect.

Accordingly, using a similar arguement as before, the form of the lateral diffusion coefficient is

$$E_y = K_y du/8.25 \tag{5.18}$$

Finally, as suggested in Equation (5.6), a background diffusion coefficient can be used to lump together the effects of unknown eddies occurring during periods of slack tide. However, introducing three different coefficients in the three coordinate directions introduces three new coefficients to be evaluated from the field data collected. This data is considered insufficient in that it is too variable to measure so many diversified effects, and it was decided to assume this coefficient was isotropic. Hence, the model forms for the longitudinal and lateral diffusion coefficients, respectively, are,

$$E_x = K_x du/8.25 + E_o \tag{5.19}$$

and

$$E_y = K_y du/8.25 + E_o \tag{5.20}$$

where

E_o = background diffusion coefficient, (L^2/T).

5.3 Vertical Diffusion Coefficient

The theory that was used to obtain the longitudinal and lateral diffusion coefficients in Section 5.2, cannot be used here throughout the vertical water column except in rare cases in which the vertical velocity profile is fully logarithmic. However, this is hardly ever the case due to flow reversals from wind shear and density currents associated with saline wedges. Taylor and Elder's theory will only apply to a small region near the bed of the channel in which the flow can be considered logarithmic. Also the parabolic distribution examined by Jobson and Sayre [1970] cannot be used, as it is also based on a logarithmic vertical velocity profile.

As this theory breaks down, and because there is not enough data to derive an empirical formula for the vertical diffusion coefficient, another form must be selected based on some form of mixing length theory. Having rejected the "4/3 power law" approach because it has been found to be invalid near solid boundaries [Zeidler, 1976], a basic approach such as Prandtl's mixing length theory is considered in which the vertical diffusion coefficient can be written,

$$E_z = K_z \ell^2 \left|\frac{du}{dz}\right| + E_o \tag{5.21}$$

where

K_z = vertical dimensionless diffusion coefficient.

Considering a layered flow because of the discrete cell structure of the numerical solution technique, Equation (5.21) can be integrated over the thickness, d_ℓ, of the layer k, from $z = d_1$ to $z = d_2$, to produce the vertical dispersion coefficient,

$$E_k = K_z \ell^2 \left|\int_{d_1}^{d_2} \frac{du}{dz}\, dz\right| / d_\ell + E_o \tag{5.22}$$

which upon integration gives,

$$E_k = K_z \ell^2 |u(d_2) - u(d_1)| / d_\ell + E_o \tag{5.23}$$

This theory is now considered for the two cases in which a density gradient due to a saline wedge is firstly absent and secondly present, in a tidal flow with associated wind shear.

From Equations (4.9), (4.28) and (4.31), the vertical dispersion coefficient in layer k is given by,

$$E_k = K_z \ell^2 \left|\frac{1}{A}\frac{dV}{dt}\frac{\ln(d_2/d_1)}{\ln(29.73d/ke)} + 0.25K_w w_s^2 \cos\theta\right. \tag{5.24}$$

$$\left.|\cos\theta|\,((d_2 - d_1)(\tfrac{3}{d}(d_1 + d_2) - 2))/N_z\right|/d_\ell + E_o$$

The only remaining problem is to define the magnitude of the mixing length, ℓ.

In most studies of diffusion, or dispersion, in rivers and estuaries, this number reflects some uniform geometric length scale such as the mean tidal depth or the width of the reach. Because of the parabolic wind profile, Equation (4.28), derived in Section 4.3, whose characteristic features such as the turning value of the vertical gradient at $z = 2d/3$, and its reversal point at $z = d/3$, are based on the one-third points of the depth (Figure 4.4), a more realistic value of the mixing length for tidal flows with wind shear is one-third of the depth. Thus the model form of the vertical dispersion coefficient is written for layer k,

$$E_k = \frac{K_z d^2}{9d_\ell}\left|\frac{1}{A}\frac{dV}{dt}\frac{\ln(d_2/d_1)}{\ln(29.73d/ke)} + 0.25\,K_w w_s^2 \cos\theta\right.$$

$$\left.|\cos\theta|((d_2 - d_1)(\tfrac{3}{d}(d_1 + d_2) - 2))/N_z\right| + E_o \tag{5.25}$$

For the case of tidal flow with wind shear, and a density current induced by a saline wedge an expression for the layer averaged vertical dispersion coefficient, E_k, similar to Equation (5.25) can be derived by adding the expressions in Equations (4.69) - (4.83) to the above theory. This form of the equation satisfies Equation (5.23) 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

$$Ri = \frac{-g/\rho \ \partial\rho/\partial z}{(\partial u/\partial z)^2} \tag{5.26}$$

The vertical diffusion coefficient averaged over the layer containing the interface $\tilde{E}_k$, is then written,

$$\tilde{E}_k = E_k \phi(Ri) \tag{5.27}$$

where

$\phi(Ri)$ = function of Richardson number to be determined, (dimensionless).

A number of investigators have suggested forms for $\phi(Ri)$. Munk and Anderson [1948] wrote,

$$\phi(Ri) = (1 + \frac{4}{3} Ri)^{-1} \tag{5.28}$$

whereas, Bowden and Hamilton [1975] suggested,

$$\phi(Ri) = (1 + Ri)^{-7/4} \tag{5.29}$$

These forms were compared in a paper by Blumberg [1977] (Figure 5.2) who found that for high Richardson numbers, $\phi(Ri)$ became too small and produced unrealistic salinity distributions. Instead he used a form proposed by Obukhov [1971],

$$\phi(Ri) = (1 - 0.1Ri)^{1/2} \quad \text{for } Ri \leq 10 \tag{5.30a}$$

$$\phi(Ri) = 0 \quad \text{for } Ri > 10 \tag{5.30b}$$

by determing numerically that for Richardson numbers greater than ten there was little or no interfacial mixing. Thus the model form of the layer averaged vertical diffusion coefficient for the layer containing the saline wedge is,

$$\tilde{E}_k = E_k \, (1 - 0.1Ri)^{1/2} \tag{5.31}$$

This form was found to give satisfactory results for this model.

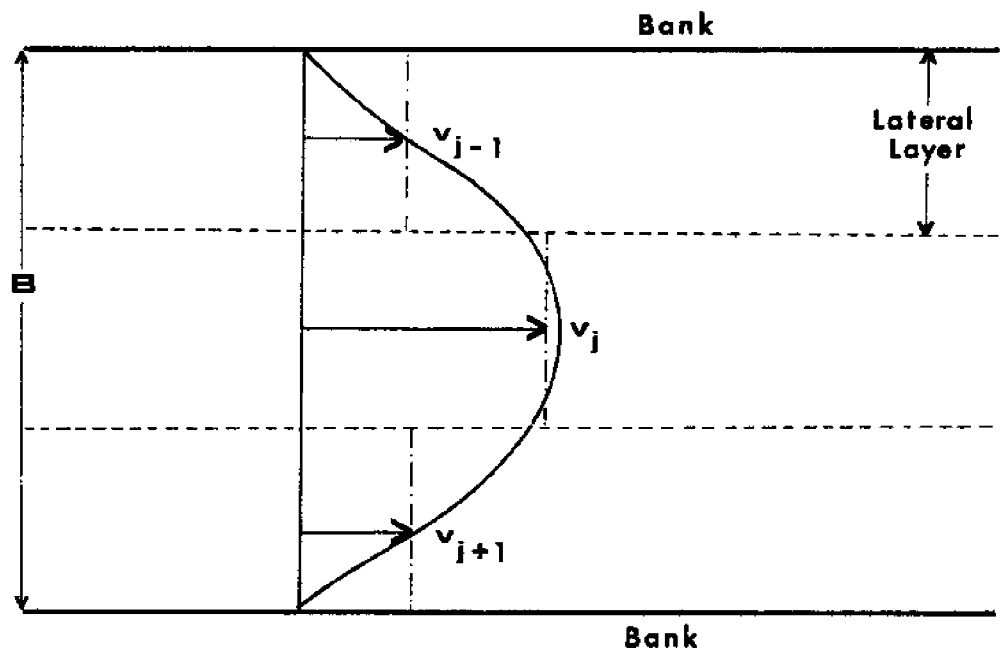


Figure 5.1 - Lateral Variation of Velocity.

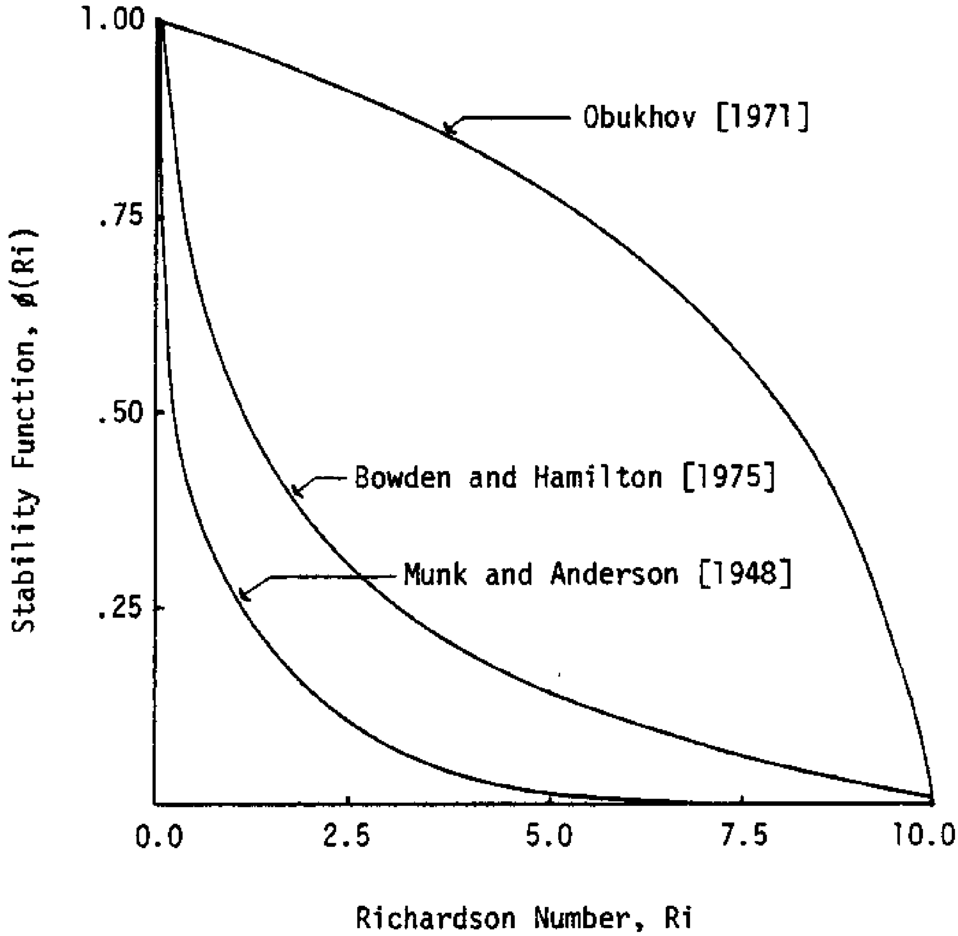


Figure 5.2 - Comparison of Saltwater Interface Stability Function.

CHAPTER 6

DEVELOPMENT OF THREE-DIMENSIONAL NUMERICAL MODEL

In Chapters 4 and 5, the governing equations were developed and arranged in a form for use in this three-dimensional model. In this chapter, these ideas and formulae are placed within a framework that will allow the user maximum flexibility in predicting transient concentration conditions in a variety of the low energy canals found along major portions of the Atlantic and Gulf coasts, while at the same time reducing the amount of 'guesswork' involved in calibrating the model, by reducing the number of model coefficients.

The aim of this study was to produce a comprehensive numerical model which is accurate, reasonably economical and relatively simple to use. This requires that the number of input parameters, excluding geometric and forcing function data, be minimized. In fact, as can be seen from the previous two chapters, the only coefficients to be read in, apart from those associated with the tidal entrance boundary condition are the dimensionless dispersion coefficients, the vertical momentum transfer coefficient and the dimensionless velocity, u_4. The solution technique is an extension of the method of second methods selected in Chapter 3.

The program, called CANNET3D (standing for CANal NETwork, 3-Dimensional), was written in FORTRAN IV for an AMDAHL 470 machine. This type of computer is the next sequence up from the IBM 370 Series, as Dr. Andahl was the former chief designer of the 360 and 370 Series at IBM. The program requires just over 200K bytes of array storage area. The program has a number of subroutines which will be referred to in capital letters as they appear in the remainder of this text. Program variables will also be referred to in the same manner.

A user's manual for the program is given in Appendix D. Appendix E consists of a flow chart for the program, whose coding is listed in Appendix F.

Before proceeding to the discussion of the model development, some of the terms that will be used are defined here. Several of the variables presented have subscripts which are a combination of the letters i, j and k. These represent spatial reference points in the x, y and z-directions respectively. Thus, i will denote the number of a segment in the reach, j will denote the lateral location from the left bank, and k will denote the layer number from the bed. If a variable has all three subscripts, then it is a function of the three independent coordinates. However, if a variable has only one or two of the 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, as it is independent of width and length.

The two superscripts used are n and n+1. These 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 theory developed can be applied at any time $n\Delta t$.

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 level n and n+1.

6.1 Layout of Geometry

The first step in numerically evaluating a proposed or existing canal network, is to arrange it for input to the model. This consists of dividing and numbering the system into reaches, junctions, dead-ends, tidal entrances, and what will be termed null points.

The *null point* arises out of the need to be able to model transport in canal networks characteristed by loops and multiple tidal entrances. Consider a system with two tidal entrances, in which the velocity field is produced entirely by changes in the tidal elevation in the receiving waterbody. Assuming that there is no phase difference between the entrances, it would be expected that the tidal inflows, from each entrance, during the flood tide, would either meet at some point, called a null point, (or more correctly an area) within the canal network, or else would recombine to flow into an upstream reach of the system (Figure 6.1). The null point is then a section within the canal network through which the net tidal flux is zero. The general formulation for identifying this type of geometry is extremely complex, requiring a knowledge of systems programming, but an individual case is usually straightforward, and a procedure will be given here to divide the system into program compatible elements.

The procedure, which is carried out at some point of the tidal cycle, usually mean tide, uses the energy equation. If a null point, as defined above (Figure 6.1a), exists, then the head loss along each series of hydraulically connected reaches and junctions to the point where the flows recombine, or to the receiving waterbody, must be equal,

$$\sum_i \Delta H_i = \sum_j \Delta H_j \tag{6.1}$$

where ΔH = head loss, (L)

i,j = reach numbers in each series of canals.

Then from Manning's equation, ignoring losses in junctions,

$$\sum_i \left(\frac{\bar{Q}_i L_i k_i^{1/6}}{A_i \; R_i^{2/3}} \right)^2 = \sum_j \left(\frac{Q_j L_j k_j^{1/6}}{A_j \; R_j^{2/3}} \right)^2 \tag{6.2}$$

in which the discharge giving the mean head loss in a reach, $\bar{Q}$, is given by integrating Manning's equation along the length of that reach,

$$\bar{Q}^2 = [Q_u^2 + Q_u \Delta Q + \frac{\Delta Q^2}{3}] \tag{6.3}$$

where Q_u = discharge at upstream end of reach, (L^3/T)

ΔQ = change in discharge along the length of the reach, (L^3/T).

For each series of reaches from a null point to the point where the flows recombine, or to the receiving waterbody, the discharge in each reach is based on the volume of the tidal prism downstream of the null point.

The location of the null point is then found by varying its location until the two sides of Equation (6.2) balance. If by following this procedure, it is found that a null point lies upstream of the junction at which the flows recombine, an alternative approach must be used. The problem is in the way a junction is matched with its upstream, downstream, left and right branch reaches. The requirement of two, or more, downstream reaches cannot easily be met because at least one of the downstream reaches will be layed out with its positive x-axis towards the junction and not away from it in the downstream sense. In other words, one of the downstream reaches must be set up as a left or right branch initially, and its cell structure will be assigned in the wrong direction. The inclusion of such a feature, desirable as it may be in some cases, was too cumbersome to include at this time, and will be left for a future version of the model.

One way in which this shortcoming might be overcome in certain situations, is if the water surface area above the junction where the flows recombine, can be divided into areas equal to the water surface area required for each downstream reach emanating from that junction, to balance Equation (6.2), and satisfy continuity,

$$\sum_{i}^{N} Q_{u_i} = A_{ws} \frac{dd}{dt} \tag{6.4}$$

Each individual water surface area can then be treated as a lake at an upward limit of the system as described below. Thus, provided the area of specific interest in a canal network with such a feature is far enough away from the junction where the flows recombine that information will not be lost to the "lakes", which are maintained at initial concentration, then the tidal prism volume capacity of the system is retained and an analysis of transport can be carried out in another area of the network.

In an analysis of a proposed design system, all the terms of Equation (6.2) are known from purely geometric considerations, except Nikuradse's equivalent sand roughness, k. As this term is raised to the one-sixth power, its variation may be considered to be small, and the procedure can continue with its cancellation from the equation.

Once the null points are established, they are 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 adjacent reaches are separately numbered, and the upstream junction number for each reach is assigned the negative value of the adjacent reach. This is to facilitate the model's handling of fluid transfer across the null point. This information is user provided to the model, by detailing the locations of reaches and junctions as described below.

Another geometric feature that sometimes occurs in canal networks is the lake. When they occur, they are usually situated at the upstream limits of the system. To model the flow in the lake would require a separate numerical model as the flow conditions are much different. However, the lake by its very presence, affects the volume of the upstream tidal prism, used in this model to calculate velocities in downstream reaches. Thus, assuming the water surface in the lake remains at the same elevation as the water surface in the remainder of the canal network, the lake is treated as a special case of a dead-end, with finite water surface area, but having no effect on mass transport to downstream reaches.

A junction of the model canal network may be either a null point described above, an area of the system at which one or two branch canals join a main canal, or else it can be a theoretical zone separating two portions of the watercourse with differing geometric features. 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 reach is assigned a number from 1 to NREACH (the total number of reaches in the system, including reaches formed by the introduction of a null point), in any order desired. This reduces the number of changes in the data with design changes. Once the required information for each reach is read in, the reaches are rearranged, retaining their original identification number, and ordered from the null points and dead-ends of the canal network to the tidal entrances. In this manner, at any junction, the upstream reaches of the junction are placed before the downstream reaches. This is done in the subroutine ORDER, and the new distribution stored in the array NRCH(*)[1]. This rearranging is done so that an incremental tidal prism volume can be calculated from the upstream limits of the system to the tidal entrance.

Note: 1. The number of asterisks in parentheses indicates the dimensional order of the array.

In a similar manner, each dead-end, except lakes, is assigned the number 1. Each interior junction (those excluding dead-ends, null points, lakes, and tidal entrances) is assigned a number from 2 to (NJUNC+1), where NJUNC is the number of internal junctions in the system. 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, and the tidal entrances are numbered from (NJUNC+NLAKE+2) to (NJUNC+NLAKE+NTES+1), where NTES is the number of tidal entrances. An example of a network numbering scheme is shown in Figure 6.2.

Once the reaches and junctions of the system have been thus numbered, the geometric data read in for each reach are its length, bottom width, mean tidal depth, left and right inverse side slopes, the numbers of the junctions upstream and downstream, Nikuradse's equivalent sand roughness, and its alignment angle. The alignment angle is to enable the x-component of the wind vector to be calculated in each reach. Similarly for each junction, its length, width, mean tidal depth, and the identifications numbers of the reaches that meet there and fractions of the upstream tidal prism volume, are read in.

Once the canal network geometry is defined, the next step is to divide the system into computational cells. Together with the input data for each reach, the number of longitudinal diversions, NDIV(*), is read in. The number of lateral and vertical divisions, NLAYY and NLAYZ respectively, are defined to be constant for the whole system. This is to ensure proper matching at junctions. At the beginning of the simulation, the depth is divided into NLAYZ layers of equal thickness. Then each layer is divided into NLAYY cells of equal cross-section area (Figure 6.3).

The junctions between reaches are simply transition zones from one reach to another, and will be used as such for every physical process modeled. To match the lateral and vertical layering of reaches entering the junction perpendicular to one another, the junction is also divided into NLAYZ vertical layers, but each layer has $NLAYY^2$ cells (Figure 6.4).

6.2 Treatment of the Velocity Field

The velocity field is obtained as a result of the various forcing functions due to the tide, wind, salinity gradients, bends, and lateral inflows. Each has a slightly different treatment which is discussed in the proceeding subsections, and in Section 6.4.

6.2.1 Tidal Velocities

As discussed in Section 4.2, the tidal velocities are generated by the rate of change of the surface elevation at the tidal entrances to the canal system. The tidal elevation data are input into the model in one of two ways. If the elevations follow a simple harmonic function, here the cosine distribution, so that the transient solution begins at slack tide, the tidal amplitude, AMP, and the period, T, are read in to form the relationship,

$$d = d_{o_{TE}} + a \cos \omega t \tag{6.5}$$

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 low tide respectively. The other method is to digitize the elevations with respect to the mean tidal elevation at each time interval.

When a data set is digitized at fixed time intervals, Δt, it is desirable not to have to alter it because stability criteria dictate that a smaller time step is required. This is true for example, if say hourly tidal elevations are collected and put on magnetic tape. The usual procedure would be to generate a larger data set based on a linear interpolation between existing data points. This may be done within the program in exactly the same way, using the user provided parameter, INTERP.

As the magnitude of the tidal velocities is directly related to the volume of the tidal prism upstream of the section of interest, the calculations begin at the dead-ends of the canal network and proceed towards the tidal entrances. This is the reason for ordering the canals in this manner in the subroutine ORDER, so that an incremental tidal prism volume can be calculated throughout the system.

The vertical velocity profile due to the tidal action alone, was assumed to be logarithmic. This 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. This logarithmic velocity is then integrated over the vertical extent of the cell, as described in Section 4.2, to form the mean longitudinal velocity in that cell.

Thus, the tidal prism volume, V_i^{n+1}, upstream of segment i in the reach during a time step, Δt, and the mean tidal velocity are known. Defining a flexible cell structure, being flexible in the vertical and lateral directions, such that the mass of fluid transported longitudinally between adjacent cells remains entirely within the receiving cell, the rate at which the cross-section area of the receiving cell varies is proportional to the velocity into it. Thus the system has two degrees of freedom, but only one governing equation. To close the system of equations for solution, the reasonable constraint is applied that the vertical layers respond to the cumulative flow from all the lateral layers as if it were one cell. Thus, the depths of each cell in any layer, except the bottom one, will always be uniform.

Considering j to be the label of a lateral cell, where j = 1, NLAYY, in the vertical layer, k, where k = 1, NLAYZ then,

$$\sum_{j=1}^{NLAYY} \sum_{k=1}^{NLAYZ} u_{jk}^{n+1} = V_i^{n+1}/(A^{n+1}\Delta t) \tag{6.6}$$

where

u_{jk}^{n+1} = velocity into cell jk, (L/T).

If the cross-sectional area of the reach, at the previous time step is A^n, the increase in area is,

$$\Delta A^{n+1} = A^{n+1} - A^n \tag{6.7}$$

Thus, considering the constraint on the vertical layers first, the change in the cross-section area of each layer, ΔA_k^{n+1}, is given by,

$$\Delta A_k^{n+1} = \Delta A^{n+1} A^{n+1} \Delta t \sum_{j=1}^{NLAYY} u_{jk}^{n+1}/V_i^{n+1} \tag{6.8}$$

Then, the changes in the cross-sectional area of any cell, ΔA_{jk}^{n+1}, in layer k is given by

$$\Delta A_{jk}^{n+1} = \Delta A_{k}^{n+1} \; u_{jk}^{n+1} / \sum_{j=1}^{NLAYY} u_{jk}^{n+1} \tag{6.9}$$

This formulation allows the center cells in a cross-section to increase in area during the flood tide as relatively more fluid passes through them, and to contract during the ebb tide. The areas of cells are uniform between adjacent cross-sections of a reach and are stored locally for the reach in ARE(*), and globally for the canal network in AREA(*,*).

Once a flexible cell structure has been defined in this manner, the problem has been reduced to a one-dimensional case of longitudinal flow between adjacent cells, exactly as developed for the one-dimensional model, which is simply one cell encompassing the entire cross-section, in Section 3.6. The method of second moments conserves the mass, C(*), center of mass, FX(*), and the width of its rectangular distribution, RX(*). Each of these three arrays is stored as a continuous string of all the cells in the network to minimize storage and improve access time.

The cells in a cross-section of a reach change, or as it will be termed from hereon, *flex*, at a uniform rate between adjacent cross-sections. However, the flexing of cells from one reach to another may be different. To deal with this problem, the junction, which is treated merely as a transition zone between adjacent reaches, is defined to have cells of equal volume which all flex at the same rate with the tide. The cells at the edge of the junction are then matched one-to-one with the cells in the immediate vicinity of the adjacent reaches without resorting to complex cell shapes and a three-dimensional form of the method of second moments. Such a form is not desirable because of the unrealistic amount of computer storage and computer time to model an admittedly ill-defined situation. The resulting numerical scheme in the junction is then an upwind difference scheme in which only the mass is conserved. However, as mentioned in Chapter 3, the numerical dispersion of this technique is proportional to the incremental length of the cell. In the junction, the cells are smaller than in the reaches because of the matching of cells from mutually perpendicular reaches meeting at the junction. Also, the surface areas of junctions are usually small compared to the total surface area of the canal network. This partially compensates for the numerical dispersion present in the junction, and the natural dispersion here is not modeled. As pointed out before, the effect of these procedures on the overall system is usually small.

Finally, it should be mentioned, that to ease the computational procedure somewhat, an extra layer of cells is added to both ends of every reach. These are called *buffer cells* (Figure 6.5), and their function is to eliminate the constant testing for junctions that is otherwise necessary for the transfer of mass to occur between adjacent cells in each reach. The buffer cells are then matched with their respective junction edge cells, and the volumes adjusted between the two.

6.2.2 Wind Induced Circulation

As with the tidal elevations, the wind data can be read 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 ft/sec and radians, respectively. Otherwise, the wind speed and direction are digitized. Again, the same INTERP parameter can be specified to be greater than one if the data needs to be provided at a smaller time step.

The velocity induced by the wind in layer k was developed in Equation (4.34) and is constant through 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 contribute to varying the cross-sectional areas of computational cells in a cross-section, because the increase in the total cross-sectional area due to its action is zero. Once the adjustment has been made to the cross-sectional areas of the cells due to the distribution of tidal velocities, the center of mass of the 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,

$$X = XT + XW(NLZ) \tag{6.10}$$

where

NLZ = number of vertical layer.

As the velocity is uniform in each layer, there is no vertical transfer of mass to conserve mass.

The junction again acts as a transition zone between adjacent reaches. A simple conservation of mass scheme is devised to match the various conditions in the adjacent cells of the adjacent reaches. This is done 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. If u_{wk} is the wind induced velocity in layer k in the positive x-direction, then the volume of fluid transport out of the dead-end layer in each time step, Δt, is,

$$V_w = u_{wk} A_k \Delta t \tag{6.11}$$

where

A_k = cross-sectional area of layer k, (L^2).

V_w = volume of fluid transported out of layer k, (L^3).

Beginning with the bottom layer, an equal volume is transferred vertically from the layer above, k+1. As the net volume flux is zero, this process is conservative.

Finally, a recommendation is made as to the choice of NLAYZ. As 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 will retain the essential features of the distribution without cancellation in the cell averaging process. In fact, for most practical simulations three layers, and probably no more than six, will provide sufficient information for the design engineer.

6.2.3 Secondary Currents

Secondary currents as defined in Section 4.4, have no effect on the longitudinal velocity distribution. They simply redistribute the fluid mass through the radial velocity pattern developed by Rozovskii [1957, Ch. 2] for a two-dimensional flow in a wide channel over a rough bed. A subroutine, HELIX, was written to describe this process.

For each layer k, at the crown of the bend, a layer averaged velocity, v_{ck}, is given from Equations (4.43), (4.48) and (4.50) by,

$$v_{ck} \;=\; 6.25\frac{d}{r}\, u_i(\overline{F}_1(k) + \overline{F}_4(k)/3.3) \qquad (6.12)$$

where

u_i = mean velocity due to tidal component in segment i, (L/T). This velocity is constant in the layer, thoughout the length of the bend, BENDL(*,*), but decays in the direction of flow to within 2 percent of zero according to Equation (4.53). For the y-axis as defined, from the left bank to the right bank, perpendicular to the longitudinal axis, specifying the radius of the bend, r, (stored in the array BENDR(*,*)), as being positive, defines flow around a bend to the left in the downstream sense. Similarly, if the bend radius is negative, the flow is around a right hand bend.

The input data to the model defines the location of the crown of the bend, the bend radius, and the length of the bend (Figure 6.6). A value for the reach number, NR, of -1 terminates the data. When a bend is located in the model, the pointer backs up against the flow to define the first longitudinal segment of the flow in which the secondary current develops (Figure 6.6). From this point, in the direction of flow, Equation (6.12) holds throughout the length of the bend, after which the radial velocity decays over the distance PL. For each longitudinal segment, denoted by the subscript i, an averaging procedure based on the length of the bend is performed giving the following cell averaged velocities in layer k, v_{ik}:

1. for segment i totally contained in the bend,

$$v_{ik} = v_{ck} \tag{6.13}$$

2. for segment i of total length x, having length Δx_1 within the length of the bend,

$$v_{ik} = \frac{v_{ck}}{\Delta x}(\Delta x_1 + \frac{1}{3}L_d(1 - \exp[-\frac{3}{L_d}(\Delta x - \Delta x_1)])) \tag{6.14}$$

3. for segment i totally outside the length of the bend, but at least partially within the decay distance, L_d, from the bend,

$$v_{ik} = \frac{v_{ck}L_d}{3\Delta x}(\exp(-\frac{3x_1}{L_d}) - \exp(-\frac{3x_2}{L_d})) \tag{6.15}$$

where

x_1, x_2 = distance 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

$$V_{ijk} = v_{ik}\Delta t \Delta x \Delta z \tag{6.16}$$

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 the 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 as for the wind induced flow case at the dead-ends, to conserve fluid mass between the vertical layers.

6.2.4 Density Currents

The circulation induced by density gradients is treated in a manner similar to the wind induced circulation in the subroutine WEDGE, except that account is taken of the longitudinal variation of the velocity in each layer using vertical transfer to conserve mass. Again, 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.

Because of the description of the motion of a saline wedge, that is of a triangular distribution entering the canal network during the flood tide and the lowering of the interface during the ebb tide as described in Chapter 4, the transient simulation to model this phenomenon must start at low tide. Thus, the initial salt wedge configuration in the network is that the elevation of the interface in each reach is the elevation of the low tide interface in the receiving waters. To model multiple tidal entrances and loops, the user must break the canal system up by determining the null points as explained in Section 6.1.

The induced velocity field is provided by the equations in Section 4.5. These velocities are then layer averaged, and the distance moved by the center of mass in each cell during the time interval, Δt, is stored in the array XS(*,*). Having adjusted the cross-sectional areas of the cells due to the distribution of tidal velocities, the center of mass of each cell is moved a distance X, similar to Equation (6.10), where X is now the sum of the component distances of the tidal velocity, the wind induced velocity, and the density current,

$$X = XT + XW(NLZ) + XS(ND, NLZ) \tag{6.17}$$

where

ND = number of segment of reach.

In longitudinally adjacent segments i-1, i say, numbered in the direction of the flow, the longitudinal velocities associated with the density current in layer k are u_{i-1k} and u_{ik} respectively. Looking at layer k in segment i, a simple fluid mass balance shows that an extra volume of fluid ΔV_{ik} is added to each lateral cell according to,

$$\Delta V_{ijk} = (u_{i-1k} - u_{ik})\Delta y \Delta z \tag{6.18}$$

where

Δy = lateral extent of cell.

Then, in each column of cells,

$$\sum_{k=1}^{NLAYZ} \Delta V_{ijk} = 0 \tag{6.19}$$

for conservation of fluid mass. Finally, the fluid mass is adjusted as in the vertical column so that the mass in each cell is conserved.

6.3 Treatment of Dispersive Terms of the Transport Equation

The common method used to treat the dispersive terms of the mass-transport equation is to develop a central finite-difference approximation to the second derivitive based on forward and backward expansions of Taylor's series about the point. Such a form was developed for the one-dimensional model in Equation (3.55), and can easily be extended to two and three dimensions [Smith, 1965, pp. 41-45; Roache, 1972]. In most problems this can be done fairly simply by applying the finite-difference approximations to all the internal points of the solution domain and then developing boundary expressions to fit the solution to the edges of the domain. These conditions vary from conservation of mass expressions, to extrapolations, to simply ignoring the dispersive term altogether at this point, and so on. The boundary specification, as artifical as it sometimes may be, is only a small part of the solution domain and if fairly well formed, does not usually impair the results too significantly.

If a numerical solution scheme is developed to model the dispersive process in a tidal canal network, using three layers laterally and three layers vertically, which is a good number for most purposes, then eight out of nine cells are boundary cells. This means that finite-difference approximations to the second derivative term can only be modeled longitudinally, or at the middle cell in each layer, either vertically or laterally. In all likelihood, the application of improvised boundary conditions in so many of the cells would lead to results whose accuracy would be questionable. Thus, a solution scheme must be found through a rearrangement of the governing equation.

Consider a one-dimensional diffusion equation in the form,

$$\frac{\partial c}{\partial t} = \frac{\partial}{\partial x}\left(E_x \frac{\partial c}{\partial x}\right) \tag{6.20}$$

where

$$E_x = \text{diffusion coefficient, } (L^2/T)$$

and introduce a variable, u_{D_i}, defined by,

$$u_{D_x} = -\frac{E_x}{c}\frac{\partial c}{\partial x} \tag{6.21}$$

Introduction of Equation (6.21) into Equation (6.20) gives,

$$\frac{\partial c}{\partial t} + \frac{\partial}{\partial x}(u_{D_x} c) = 0 \tag{6.22}$$

which is the one-dimensional convection equation in which u_{D_x} is a longitudinal velocity term. u_{D_x} is called the diffusion velocity [Spaulding, 1976].

In an exactly similar way, diffusion velocities can be defined in the y- and z-directions respectively,

$$u_{D_y} = -\frac{E_y}{c}\frac{\partial c}{\partial y} \tag{6.23}$$

$$u_{D_z} = -\frac{E_z}{c}\frac{\partial c}{\partial z} \tag{6.24}$$

such that when applied to the three-dimensional diffusion equation, written in vector form,

$$\frac{\partial c}{\partial t} = \nabla(\overline{E} \cdot \nabla c) \tag{6.25}$$

the three-dimensional convection equation is obtained,

$$\frac{\partial c}{\partial t} + \overline{u}_D \cdot \nabla c = 0 \tag{6.26}$$

where

$$\overline{u}_D = (u_{D_x}, u_{D_y}, y_{D_z})$$

The diffusion velocities can then be used to transfer mass between adjacent cells, and apply in the boundary cells because the condition of zero flux through the side of the channel is satisfied. As the velocity field has been reduced to a one-dimensional transport between longitudinally adjacent cells using the flexible grid structure defined in Section 6.2, the longitudinal dispersion term is treated slightly differently from the other two dispersion terms.

6.3.1 Longitudinal Dispersion Term

From Equation (6.21) 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,

$$u_{D_i}^{n+1} = -2\left(\frac{\Delta x_i E_{x_{ijk}}^{n+1} + \Delta x_{i+1} E_{x_{i+1jk}}^{n+1}}{\Delta x_i c_{ijk} + \Delta x_{i+1} c_{i+1jk}}\right)\left(\frac{\Delta x_{i+1} c_{i+1jk}^n - \Delta x_i c_{ijk}^n}{\Delta x_i + \Delta x_{i+1}}\right) \tag{6.27}$$

where

Δx, Δx_{i+1} = lengths of segments i and i+1 respectively.

$E_{x_{ijk}}^{n+1}$ = dispersion coefficient in cell ijk, (L^2/T).

Thus, in an analogous manner to the distances associated with the velocity terms in Section 6.2, a longitudinal diffusion distance, XD, or more correctly, a dispersion distance as the coefficient is cell integrated, is defined as

$$XD = u_{D_x} \Delta t \tag{6.28}$$

and added to the other distance terms of Equation (6.17), to give the final model displacement of the center of mass of cell ijk in the time interval Δt as,

$$X = XT + XW(NLZ) + XS(NR, NLZ) + XD \tag{6.29}$$

6.3.2 Lateral and Vertical Dispersion Terms

As there is no specific lateral or vertical velocity term 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 (6.23) and (6.24) respectively, which transfer fluid between adjacent cells simply conserving mass. Thus, in the lateral and vertical directions a straightforward upwind differencing technique is used.

From Equation (6.23), the mass flux, cu_{D_j}, between lateral cells j and j+1, due to lateral dispersion, is written in finite-difference form

$$cu_{D_j}^{n+1} = -\frac{1}{2}(E_{y_{ijk}}^{n+1} + E_{y_{ij+1k}}^{n+1})(c_{ij+1k}^{n} - c_{ijk}^{n})/\Delta_{y_{ijk}} \tag{6.30}$$

This transfer occurs in the direction of the vector lateral dispersion velocity, u_{D_j}, which is directly related to the lateral concentration gradient.

Similarly, from Equation (6.24), a mass flux, cu_{D_z}, between vertical layers k and k+1, due to vertical dispersion, is given as,

$$cu_{D_k}^{n+1} = -\frac{1}{2}(E_{z_{ijk}}^{n+1} + E_{z_{ijk+1}}^{n+1})(c_{ijk+1}^{n} - c_{ijk}^{n})/\Delta z_k \tag{6.31}$$

Again, transfer occurs in the direction of the regative 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, as defined in Section 5.3. Even though the saltwater wedge is modeled as a sharp interface, dispersion can be introduced across this interface by defining some distance over which the density varies from ρ_o to $\rho_o + \Delta\rho_c$. An obvious choice is the thickness of a vertical layer Δz as an approximation. This is a somewhat crude assumption, but better than assuming no dispersion across the interface at all. Also, field measurements indicate that the thickness of the transition zone varies considerably during the course of a tidal cycle.

Thus, from Equation (5.26), the Richardson number, Ri, is given by,

$$Ri = \frac{-g/\rho_{av}\,\Delta\rho_o/\Delta z}{(du/dz)^2} \tag{6.32}$$

where

ρ_{av} = average density between the two layers, (M/L^3).

Equation (6.32) is given explicity, as du/dz can be calculated exactly from the various superimposed expressions for the velocity field developed in Chapter 4. The function $\phi(Ri)$ defined by Equation (5.30), is then used to modify Equation (6.31), provided the interface lies within one-half of a cell's depth from the plane separating vertically adjacent cells, giving,

$$cu_{D_k} = \frac{1}{2}(1 - 0.01Ri)^{1/2}(E_{z_{ijk+1}} + E_{z_{ijk}})(c^n_{ijk+1} - c^n_{ijk})/\Delta z_k$$

$$\text{for} \quad Ri \leq 10 \tag{6.33a}$$

$$cu_{D_k} = 0 \qquad \text{for} \quad Ri \geq 10 \tag{6.33b}$$

6.4 Lateral Inflows

The rate of production or loss term, r_p, introduced in the three-dimensional mass-transport Equation (3.13), was defined to have the distribution given by Equation (3.16) for the one-dimensional model. This form can be extended to the three-dimensional model by defining $q_{I_{ijk}}$ to be the lateral inflow per unit length of cell ijk. If the concentration of this inflow is $c_{I_{ijk}}$, then the rate of production or loss term, $r_{p_{ijk}}$, is,

$$r_{p_{ijk}} = q_{I_{ijk}} c_{I_{ijk}} / A_{jk} \tag{6.34}$$

The lateral inflow step is performed as a simple addition of mass to cell ijk, and is done before the convection step. This is 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 concentration of the substance carried in by the lateral inflow is added to the concentration of the substance already in the cell,

$$\tilde{c}^{n+1}_{ijk} = c^n_{ijk} + q^{n+1}_{I_{ijk}} c^{n+1}_{I_{ijk}} \Delta t / \Delta y_{ijk} \, \Delta z_j \tag{6.35}$$

where

$\tilde{c}^{n+1}_{ijk}$ = resulting concentration in cell ijk after lateral inflow addition, (dimensionless)

c^n_{ijk} = concentration in cell ijk at previous time level n, (dimensionless)

Δx_i = uniform longitudinal length of cell in segment i at reach, (L)

Δy_{ijk} = lateral width of cell ijk, (L)

Δz_k = vertical thickness at layer k of reach, (L).

Then, the sum of the lateral inflows in segment i, is added to the upstream tidal prism volume, V_i^{n+1}, to give

$$\tilde{V}_i^{n+1} = V_i^{n+1} + \sum_{j+1}^{NLAYY} \sum_{k=1}^{NLAYZ} q_{I_{ijk}} \Delta t \, \Delta x_i \tag{6.36}$$

where

$\tilde{V}_i^{n+1}$ = tidal prism volume with lateral inflow added, (L^3).

This value of the tidal prism volume is used in the theory presented in Sections 6.2.1 and 4.2, to determine the cell averaged tidal induced velocities, and the change in the cross-sectional areas of cells in each segment.

The advantages of expressing the lateral inflow in this manner are that the resulting model formulation is somewhat simplified. If the effect of the lateral inflow were followed exactly, then it would be necessary to model the lateral circulation produced by the inflow, to both conserve mass, and the horizontal water surface assumption. In the first place, the quality of the data cannot be expected to reflect this mechanism, and secondly the lateral inflow would never be uniformly distributed throughout the length and depth of the cell.

Another advantage is that river flows, or flows over salinity structures, banks, etc. or sinks, can be readily modeled by specifying the cells into which the inflow or outflow occurs. As the concentration is added before the convective step, the inflow conditions remain in phase with the solution scheme, which means that inflow concentrations are convected with induced velocities at the same time level, and not the next.

In the model, the lateral inflow can be treated in one of two ways. If the inflow's rate and concentration remain constant throughout the simulation, then for each such inflow, OPT2(*) is given the value 0, and the reach number, NR, the segment NDX, the number of the cell in the cross-section, NC, (based on the coordinate system described in Section 6.1), the lateral inflow rate, QIC(*) and the concentration, CIC(*), are read in initially and are stored for the run. If the lateral inflow and/or its concentration are variable, OPT2(*) is given the value 1 and only the variables NR, NDX and NC are initially read to determine its location, and are stored in the array LOCV(*) in the subroutine ORDER. OPT2(*) = -1 terminates the data. During the simulation, the variable inflow data are read in at each time interval at which tide or wind data is specified (the INTERP variable will calculate intermediate values as before). However, it should be noted that the inflow data <u>must</u> be in the same order as given initially.

6.5 Decay Coefficients

The three-dimensional mass-transport equation, given in Equations (3.13) and (5.1), describes the transport of a conservative, passive substance in a velocity field (u, v, w). The term conservative means that the concentration of the substance does not decay due to outside stimuli such as sunlight, absorption by bank or suspended solids, or reactions with other substances found in the flow field.

If, for example, dissolved oxygen (DO) were being modeled on its own without an interactive process with ultimate oxygen demand (UOD), or nitrites without the conversion process to nitrates, and so on, a decay term could be added at locations where these processes were known to exist to model the phenomena. Similarly, a source term could be added at locations where the substance is produced as described in Section 6.4.

There are many forms of decays which are usually termed first order, second order, or whatever best fits the process. The most common form, however, is the first order exponential decay described by,

$$\tilde{c}_o = c_{RW} + (c_o - c_{RW}) \exp(- Kt) \qquad (6.37)$$

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

$$\frac{\partial c_o}{\partial t} = - Kc_o \qquad (6.38)$$

As Equations (5.1) and (6.38) are linear, they may be superimposed to produce the three-dimensional mass-transport equation for a noncenservative substance,

$$\frac{\partial c}{\partial t} + \frac{\partial}{\partial x}(uc) + \frac{\partial}{\partial y}(vc) + \frac{\partial}{\partial z}(wc) = \frac{\partial}{\partial x}(E_x \frac{\partial c}{\partial x})$$

$$+ \frac{\partial}{\partial y}(E_y \frac{\partial c}{\partial y}) + \frac{\partial}{\partial z}(E_z \frac{\partial c}{\partial z}) + r_p - Kc \tag{6.39}$$

In the model, the distinction is made between two different forms of decay, one which exists throughout the volume of a reach, and one that only exists in certain localized portions of the reach. The reach uniform decay coefficient, K, in Equation (6.37), is read in with the other reach data and is stored in the array RDECAY(*). The localized decay coefficients, which are assumed to remain constant throughout the period of the simulation, are input in a similar manner to the lateral inflows. For each coefficient, its reach number, NR, segment, NDX, cell number in cross-section, NC, and decay coefficient, DECAY(*) are read in while the value of OPT for each set of data is nonnegative. A negative value terminates the input.

It is not convenient to model the decay during the convective step in an explicit mass-in-cell technique, such as the method of second moments, because of the increased complexity introduced to follow the various portions of the volume as it is exchanged between adjacent cells. It is easier to model the decay over a time interval Δt, either before or after the convective steps are performed. In this model, the decay step immediately follows the convective step, and a finite-difference scheme, for simplicity, is used to approximate Equation (6.38) in each cell, ijk, in the form

$$c_{ijk}^{n+1} = c_{RW} + (K_R + K_{ijk})(\tilde{c}_{ijk}^{n+1} - c_{RW})\Delta t \tag{6.40}$$

where

K_R = reach constant decay coefficient, (1/T)

K_{ijk} = cell constant decay coefficient, (1/T)

$\bar{c}_{ijk}^{n+1}$ = concentration in cell after convective step, (dimensionless).

6.6 Boundary Conditions

As discussed in Section 3.1.5, the three-dimensional mass-transport equation is a second order parabolic equation, being first order in time, t, and second order in the spatial variables (x, y, z). To close the numerical scheme for solution requires one set of initial conditions at all points in the canal network, and specifying conditions on all physical boundaries of the solution domain.

In the same manner as for the one-dimensional model, the initial conditions are either read in (OPT4 = 1), or are generated within each reach to equal the background concentration, c_{RW}, (OPT4 = 0).

For each solid boundary, the dead-ends, the banks, the bed and the air/sea interface at any time, the condition of zero mass flux is applied. One of the advantages of the model formulation presented in the preceeding sections of the chapter, is that the numerical scheme automatically satisfies this conditon everywhere, except at the tidal entrances which will be treated separately. Even for the dispersion terms which usually require a boundary condition to replace the spatial second derivative at this point, a dispersion velocity was derived which also satisfies this requirement.

Thus, the only boundary conditon which requires some deeper consideration is the tidal entrance condition. The condition adopted for the model was a revised form of the boundary condition used for the one-dimensional model. Unfortunately, unlike for the one-dimensional model, the flow at the tidal entrance cannot be divided into mass inflow during the flood tide, and mass outflow during the ebb. This is because the circulation patterns produced by wind and salinity gradients may cause flow reversals even at the mouth of the canal networks giving both inflow and outflow from different cells at the same time.

To model this region, it was assumed that once the flow exits the system, it becomes vertically well mixed, and that the resulting concentration decays to the background concentration as described for the one-dimensional model in Equation (3.26). In the model, a cell is generated having a volume equal to that of the last segment in the tidal entrance reach. If the concentration of this cell is $\tilde{c}^{n}_{TE}$ and the volume is V^{n}_{TE}, then the resulting concentration at the end of a convective step, c^{n+1}_{TE}, is given by,

$$\tilde{c}^{n+1}_{TE} = \frac{\sum_{j=1}^{NLAYY} \sum_{k=1}^{NLAYZ} V^{n+1}_{jk} c^{n}_{jk} + (V^{n+1}_{TE} - \sum_{j=1}^{NLAYY} \sum_{k=1}^{NLAYZ} V^{n+1}_{jk}) c^{n}_{TE}}{V^{n+1}_{TE}} \tag{6.41}$$

where

V^{n+1}_{jk} = volume of fluid transport out of reach from a cell, (L^3)

c^{n}_{jk} = concentration of 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 so that fluid of concentration c^{n}_{TE} enters the cell, so that the width of the volume in the cell becomes equal to the width of the cell, Δx_i,

$$c_{ijk}^{n+1} = R\tilde{c}_{ijk}^{n+1} + (1 - R)c_{TE}^{n} \tag{6.42}$$

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 c_{TE}^{n+1} is formed in the tidal entrance cell, its value is assumed to decay towards the background concentration, c_{RW}, described by

$$c_{TE}^{n+1} = c_{RW} + (c_{TE}^{n+1} - c_{RW}) \exp(-3\Delta t/\tau) \tag{6.43}$$

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 conditon was found to work satisfactorily, and the comments made in Chapter 3 as to the variability of the coefficient, τ, apply here also, to some degree.

6.7 Results Presentation

The presentation of results is done in three phases, which will be referred to as input data, digital results, and graphical results.

The imput data section lists the parameters which are read into the model. After giving the test number, the global parameters such as those associated with the time interval, the tidal amplitude, AMP, the background concentration, c_{RW}, the dispersion coefficients and parameters detailing the layout and cell structure of the canal network are listed. Following this comes a listing of all the junction parameters, tidal entrance data, lateral inflow data, decay data, bend data, and finally the geometric data for each reach. A typical output showing the listing of input parameters can be seen in Appendix G, for the 57 Acres study canal.

Once these data are listed, the transient solution begins, after specifying the type of problem being investigated, in terms of wind, harmonic or digitized tidal elevations, and so on. Following this, the initial data, in digital form, for concentrations and velocities are listed (the velocities are set to zero initially). First, the time, tidal elevation with respect to mean sea level, the wind speed and direction, are given. Next the reach number is printed, immediately followed by values of concentration and then velocities in that reach. The first line gives the concentration values for the upstream segment listed in order from ((j=1, NLAYY), k=1, NLAYZ) across the line. Continuation occurs where necessary. The next line lists the velocity values for these cells in the same order. In this manner, each segments' values are printed down to the end of the reach, at which point the values for the next reach are listed. Once the data for all NREACH reaches have been printed, the values for the junctions (concentrations only) are listed in the same manner.

The user controls the number of time intervals between writing by the parameter NPRINT. At every NPRINT time steps, the values for time, tidal elevation, wind speed and direction, concentrations, and velocities are listed as before. When the simulation is completed, the final results are also printed out. A sample output is given in Appendix G.

A second user controlled parameter is NPLOT. If NPLOT IS specified to be greater than the total number of time steps, then no graphs are generated. If a lower value is given, then every NPLOT time steps, two graphs are generated for each reach in sequence. The left hand graph gives plot of the concentration values versus distance for each layer, and an average plot over all the layers. The right hand plot shows the same results for the velocities. To keep the plot within bounds, the user must specify the maximum concentration expected, CMAX, and the maximum velocity anticipated, VMAX, so that axial values can be printed. The plots are generated from the upstream section using the parameter DXMIN (the length of the smallest segment in the network), and the results shown for each distance increment, DX(*). A scale is provided between the two plots at DXSC increments. A sample output showing these features is given in Appendix G.

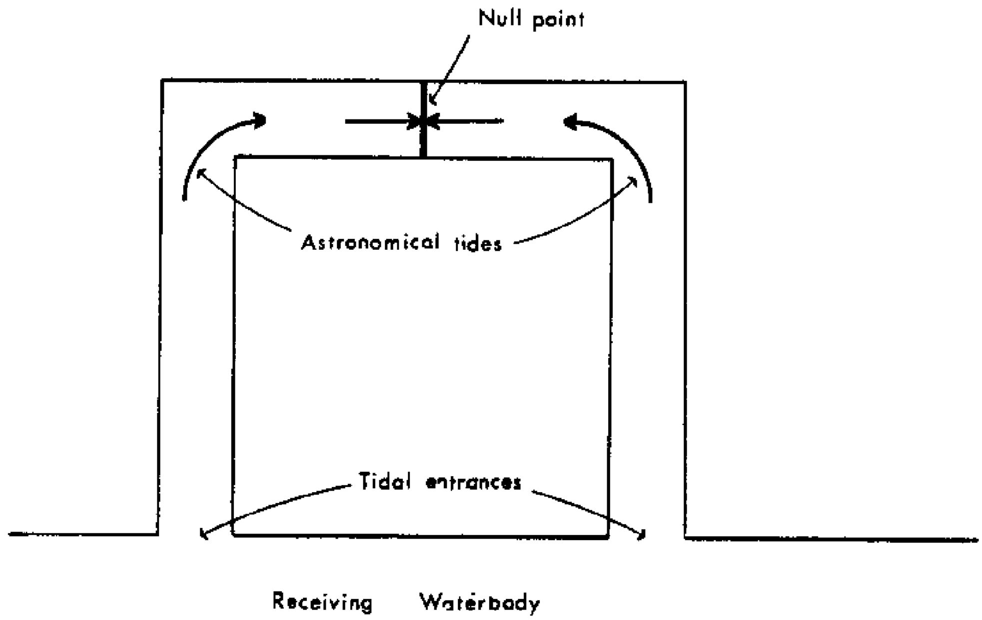


Figure 6.1a - Astronomical Tides Meeting at Null Point.

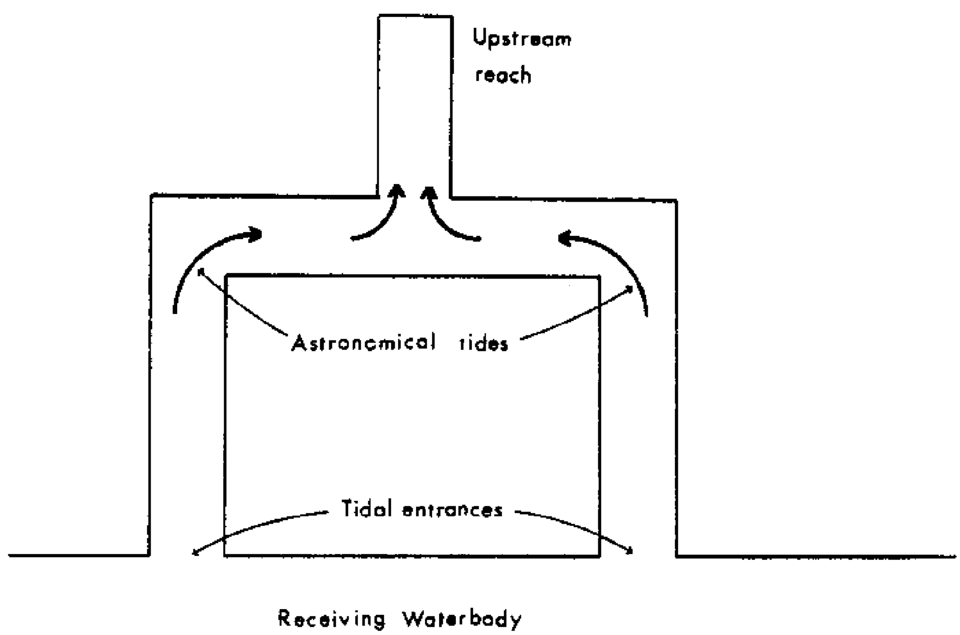


Figure 6.1b - Astronomical Tides Recombining at Upstream Junction.

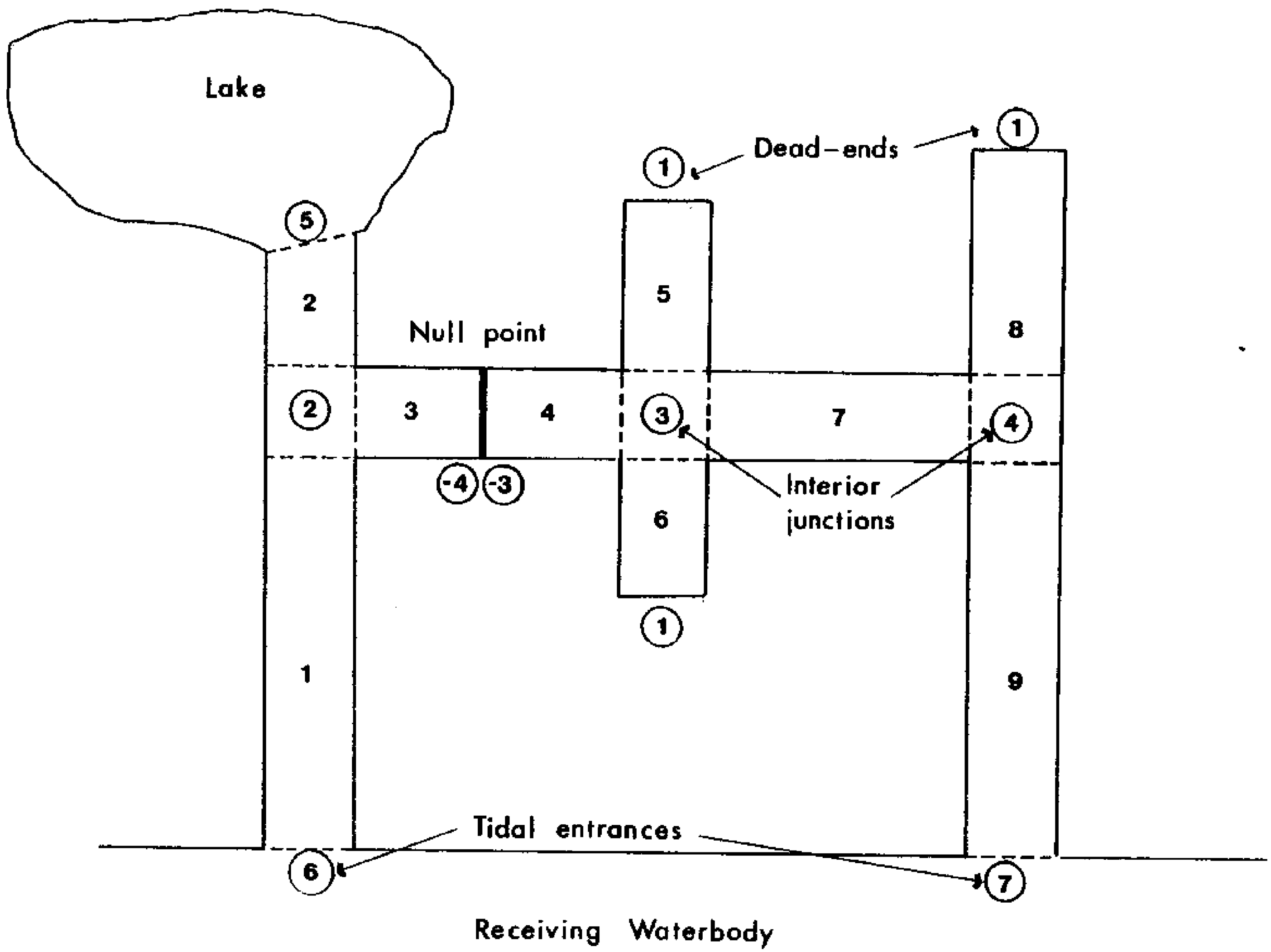


Figure 6.2 - Schematic Layout of Canal Network Showing Features.

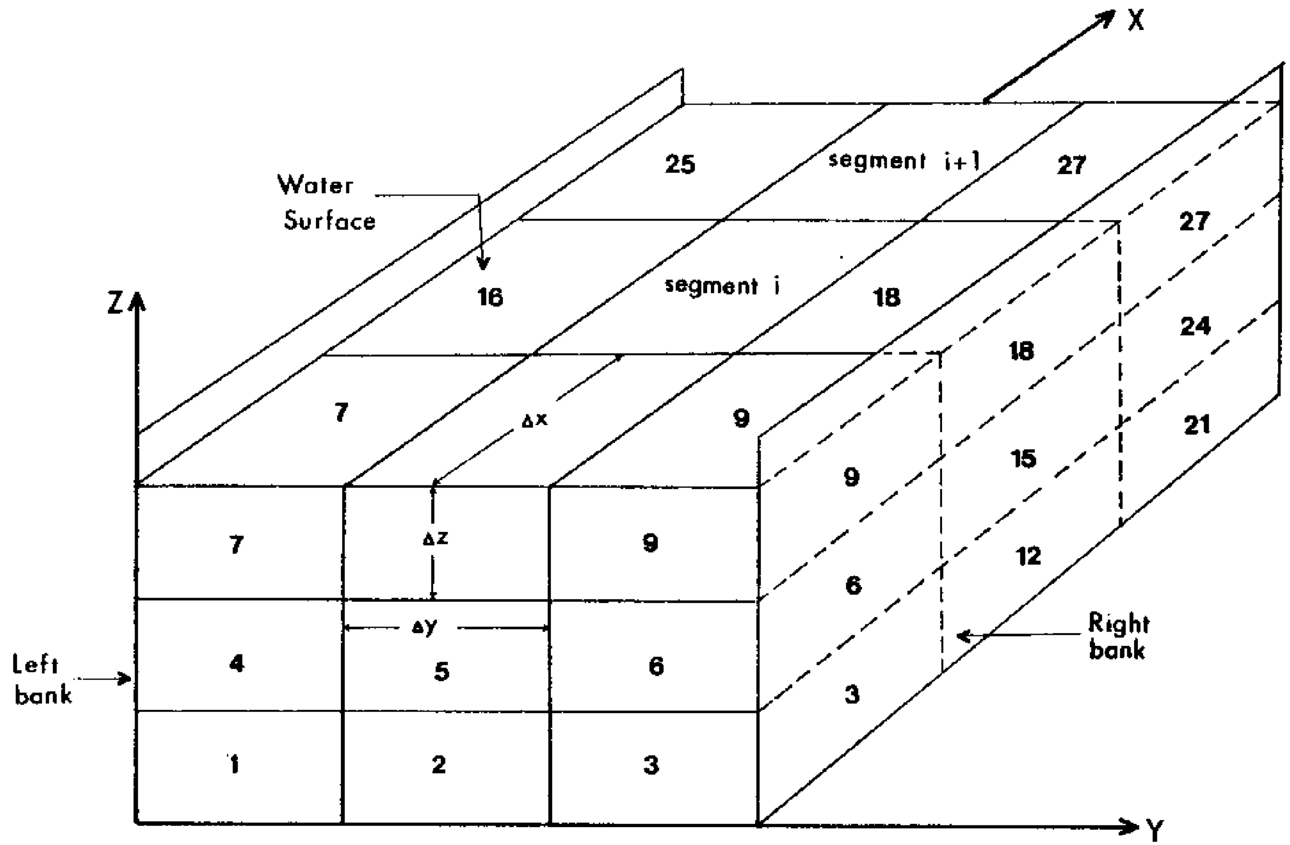


Figure 6.3 - Cell Structure in Reach.

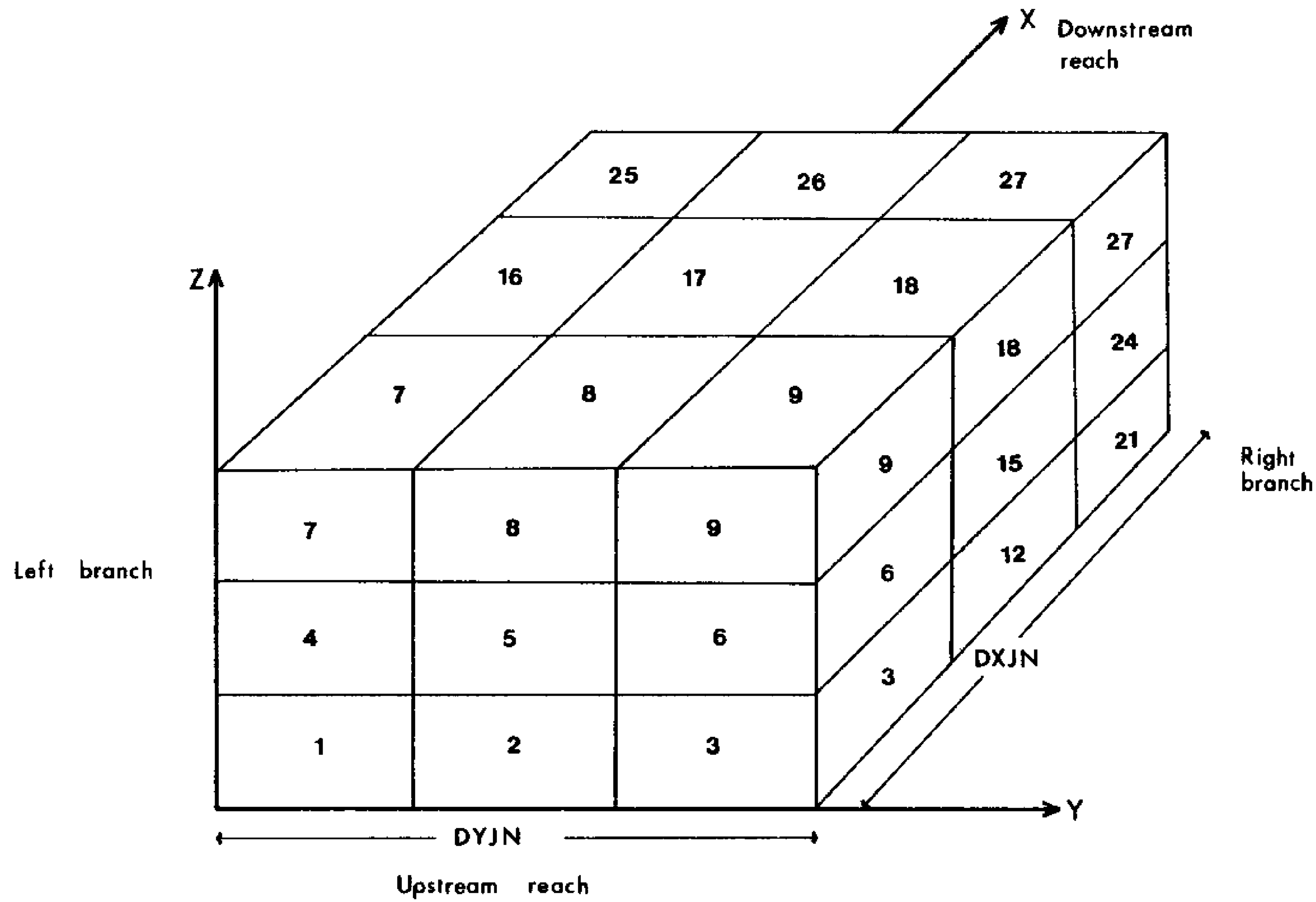


Figure 6.4 - Cell Structure in Junction.

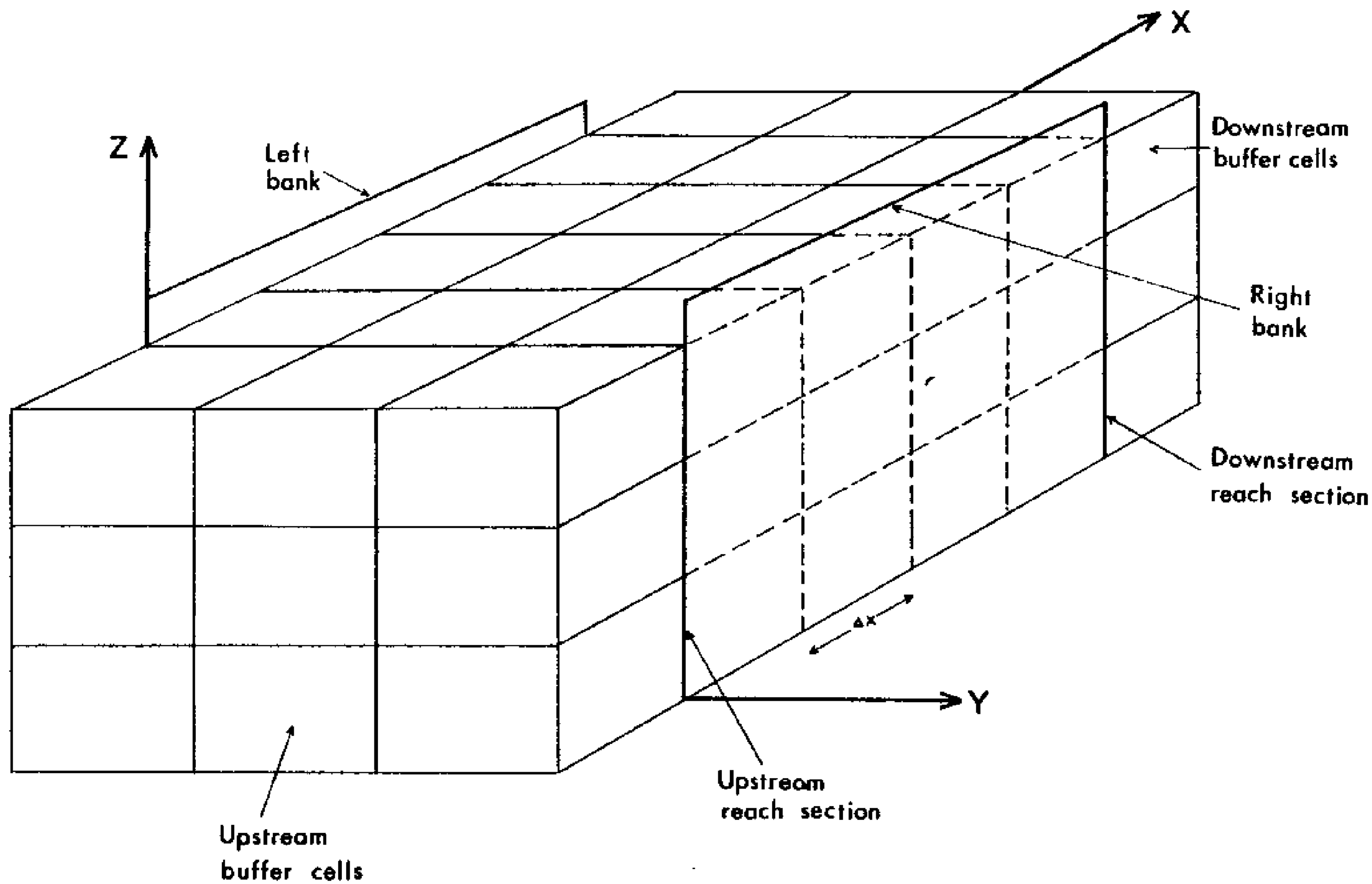


Figure 6.5 - Buffer Cell Structure.

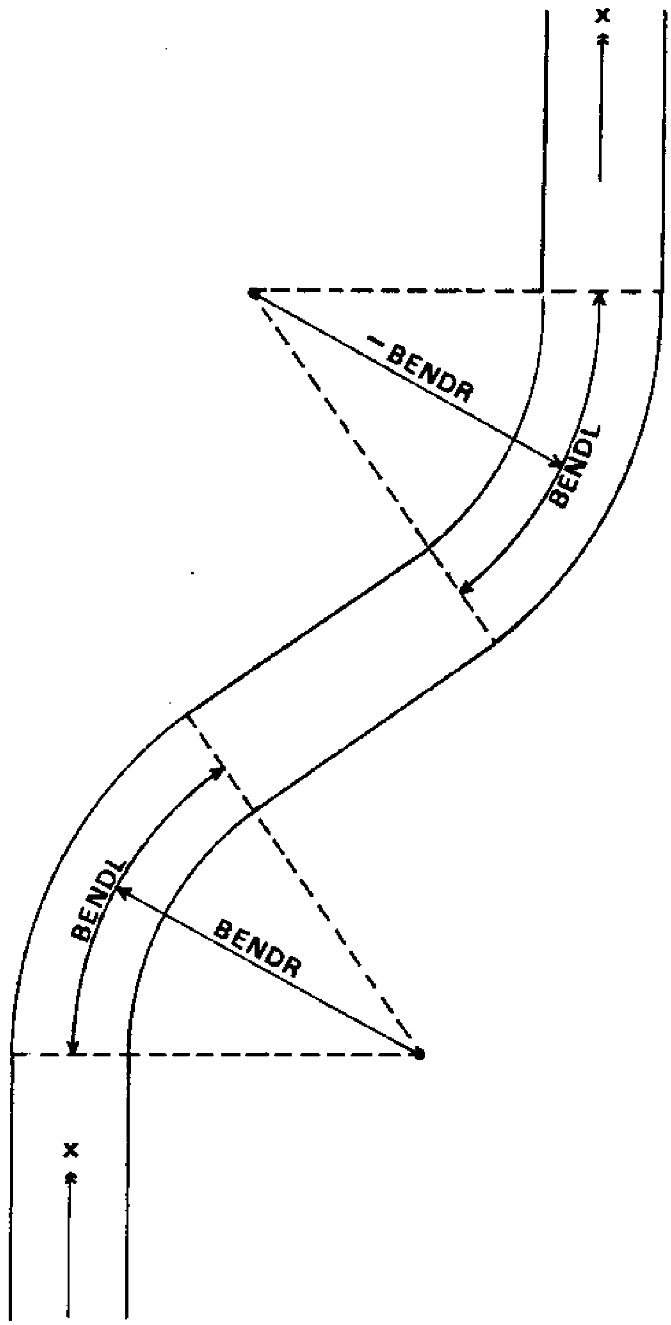


Figure 6.6 - Schematic Layout of Bend.

CHAPTER 7

MODEL STABILITY AND CONVERGENCE CRITERIA

7.1 Conservative Property, Order of Accuracy and Transportiveness

The method of second moments has been used to develop a model to simulate mass transport in low energy tidal canals. The method is conservative and second order accurate. The conservative property, as discussed by Roache [1972], ensures that the accumulation of mass within the solution domain equals the net flux across the boundaries during both the convection and dispersive processes, as a dispersion velocity term has been used to model the second order terms. Changes then, in the domain, should only occur due to computer round-off error, particularly in the square-root term which is used at each time step and in each cell to calculate the width of the rectangular distribution in the cell.

Most common finite-element and finite-difference methods that have been used to model the mass-transport equation, are first order accurate. This is because, as shown in Chapter 3, there is a second order error, termed numerical dispersion, which results from the approximation to the first-order time derivative, and to the first-order spatial derivative, if the forward or backward-difference methods are used. As the method of second moments, and the second upwind difference scheme with limited antidispersion and flux corrected transport [Lee, 1977], discussed in Chapter 3, eliminate the numerical dispersion, the methods are second order accurate.

The order of accuracy of the numerical scheme is not usually a problem. In many examples, such a formulation is used to model mass transport in rivers and estuaries, in which the natural dispersion being modeled is far greater than the numerical error being produced by the model for reasonable choices of the time and spatial inenements, Δx and Δt respectively. Bella and Dobbins [1968], however, did employ a correction factor based on a theoretical form of the numerical dispersion and subtracted the numerical dispersion coefficient from the natural dispersion coefficient in the model. This technique worked well for them because the resulting coefficient was still positive, and positive dispersion has a smoothing effect on the results.

Unfortunately, in low energy tidal canals such as those investigated in Florida, and indeed which occur throughout the Gulf of Mexico and Eastern United States, dispersion coefficients are very small, maximum coefficients typically being less than 5 ft^2/sec. This has two effects. Firstly, the numerical dispersion ignored by many investigators in their models, may now be larger than the dispersion being modeled, giving results which are either unstable or severely attenuated. The instabilites arise when the inherent numerical dispersion is negative. Negative dispersion acts in exactly the opposite way from the smoothing effect of positive dispersion. Secondly, correction factors, such as those used by Bella and Dobbins, may also give a resultant negative dispersion coefficient, and hence instabilities may again arise.

Lee [1977] used this correction technique, but controlled it using flux-corrected transport [Boris, Book and Hain, 1973, 1975, 1976]. The method required that no new local maxima or minima be introduced into the solution, and it worked well. Unfortunately, it suffered from the drawbacks that the formulation, particularly in junctions, was cumbersome and not physically intuitive.

The method of second moments does not suffer from these disadvantages because there is no numerical dispersion to correct for. As a result, the technique is more clearly expressed in a way that is physically meaningful, as it simply conserves the moments of the distribution of each cell about the center of mass in that cell. This leads to a simpler model when dealing with a system as complex as a three-dimensional canal network.

Finally the solution scheme, as arranged in this model, is transportive in every step. Roache [1972] discusses this property by saying that "the effect of a pertubation in a transportive property is advected only in the direction of the velocity". Upwind difference methods such as the technique developed by Lee [1972] for the convective step, and the method of second moments process this property. No finite-difference formulation of the convective process using space-centered derivatives for the convective term is transportive. In this model, however, because the three-dimensional dispersion terms are treated using a dispersion velocity, the model is also transportive in the second order terms.

7.2 Stability Criteria

Stability is generally defined to be a property associated with the numerical scheme's ability to damp out with time small pertubations in the solution. These pertubations are caused by truncation errors of the infinite series approximation to the governing equations, and by round off errors in the finite word length of computer storage. Usually, this second error is much smaller that the first one, but because of the large number of square roots calculated, a very inefficient algorithm on some computers, this second error may be important. The second error type is also called random error, because its effect is virtually impossible to analyze.

If "round-off" errors, e_{ijk}, are introduced into the numerical solution scheme in each cell ijk, such that the exact difference solution, $F(\Delta x,\Delta y,\Delta z,\Delta t)$, is approximated by the numerical solution, $\tilde{F}(\Delta x,\Delta y,\Delta z,\Delta t)$, and if each

$$|e_{ijk}| < \varepsilon \tag{7.1}$$

where ε = some small number, then if $|F-\tilde{F}|\to 0$, as $\varepsilon\to 0$, the scheme is said to be stable [Smith,1965]. Otherwise the scheme is unstable.

Such a definition is useful because it can give a guideline for analysing a numerical scheme. However, it is possible that the above condition may not be met, but that the scheme still gives a good approximation to the correct result. This might be the case if the errors accumulated very slowly, or else cancelled each other out. It is often the case in a stability analysis that a scheme is programmed and the results compared with an analytic solution to see whether good agreement is found. If there is good agreement, the model is said to be stable.

In some numerical schemes, it is possible to use a conventional stability analysis, such as the discrete pertubation stability analysis or the von Neumann stability analysis [Roache, 1972, pp. 36-53], to examine the expected accuracy. Usually such an analysis can only be based on a simplified form of the governing equations, as an extension to include extra terms makes the analysis too difficult. Thus, in most cases, stability criteria are developed that make sense physically for the problem being investigated, and these are adjusted in view of the model results obtained.

Such an investigation, performed for the one dimensional model developed in Chapter 3, will be extended here to include all three coordinate directions. The criteria are divided into velocity criteria and dispersion criteria, and are examined in the following two subsections.

7.2.1 Velocity Criteria

The simplest and most obvious velocity criteria is that pollutant mass should only be transferred between adjacent cells during one time step. The model is set up this way, so that if too large a velocity is present, such a transfer of mass will probably cause an error in the square root term associated with the width of the distribution in a cell, because this width is now negative. In its simplest form, this condition can be written,

$$|u_{max\,ijk}| < \frac{\Delta x}{\Delta t} \tag{7.2}$$

where $u_{max\,ijk}$ = largest velocity expected in cell ijk, (L/T).

Before setting up the model, then, the condition expressed by Equation (7.2) should be examined in each proposed cell of the model, particularly in junctions. However, this would be both cumbersome and unnecessary if the condition in only the most critical cells of each reach and junction can be examined. To select these cells, the type of velocity field expected must be considered, and points of maximum anticipated velocities identified.

For the case of an astronomical tide alone, the maximum velocity in each reach and junction is expected at the downstream end since the tidal velocity is a linear function of distance from the dead-end. Also, as the vertical velocity distribution is assumed to be logarithmic, based on the mean depth of flow in each cell, the maximum velocity in the vertical will occur in the middle cell of the top layer. Thus, the critical cell is the middle cell in the top layer at the downstream end of the reach or junction, assuming the length of each segment, Δx, is either constant, or does not vary significantly.

For wind induced circulation alone, the maximum velocity is at the water surface. Thus, the critical cells will be those in the top layer of each reach.

For density currents alone, induced by 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 between one or more cells. Thus, one cell in each vertical layer at the downstream end of each reach should be examined for this case.

The remaining two components of the velocity field are the induced circulation in bends, and the flow due to the volume rate of lateral inflow. However, unless a river is modeled, using the lateral inflow capability, these two effects produce velocities whose magnitudes are small compared with other velocities present, and hence they will not be included in the following analysis.

When this model is run, it will usually be for one of two combinations of conditions, and these will be examined separately. For the case of wind induced circulation superimposed on an astronomical tide, the critical cell is the middle cell in the top layer at the downstream end of each reach. From Equation (4.30), the maximum wind induced velocity expected, u_{wmax}, can be expressed of a function of the maximum component of the wind $|w_s \cos \Theta|_{max}$ along the longitudinal axis of the reach,

$$u_{wmax} = K_w |w_s \cos \Theta|^2_{max} (d - \Delta z)^2 / 4dN_z \qquad (7.3)$$

where Δz = thickness of top layer, (L).

A similar expression can be developed for the maximum astronomical tide component of the velocity field, u_{tmax}, from Equation (4.12),

$$u_{tmax} = \left(\frac{1}{A} \frac{dv}{dt}\right)_{max} \left(\frac{d}{\Delta z} \ln \frac{d}{d-\Delta z} + \ln \frac{29.73(d-\Delta z)}{ke} / \ln(29.73d/ke) \right) \qquad (7.4)$$

Thus for this case, the velocity stability criterion is given from Equations (7.2), (7.3), and (7.4), as,

$$|u_{wmax}| + |u_{tmax}| < \frac{\Delta x}{\Delta t} \tag{7.5}$$

This form assumes that either Δx is constant throughout a reach, or else its variation is small (particularly if the variation is to increase its value from that at the downstream end of the reach). If Δx does vary throughout the reach, it may be necessary to examine the criterion of Equation (7.5) at other top-center cells in the reach. Merely checking the stability condition at the tidal entrance can be dangerous if a wind induced circulation is present, because the criterion may be violated in an upstream reach whose channel is aligned closer to the direction of the prevailing wind. When a wind is blowing, its induced flow is usually greater than the flow produced by the astronomical tide, and care must be taken to check conditions carefully in all reaches.

For the case in which a density current exists due to the motion of a saline wedge, the vertical velocity profile within the saltwater wedge during the flood tide is given in Equation (4.69), before the wedge impinges on a dead-end of the canal network. In the top layer of the saltwater wedge, the maximum velocity, u_{smax}, is given by,

$$u_{smax} = \frac{u_4}{\Delta z} \; d_s \left(\frac{dd_s}{dt}\right)_{max} \left(1 - \left(1 - \frac{\Delta z}{d_s}\right)^3\right) \tag{7.6}$$

where Equation (7.6) is applied at the downstream section of each reach. This expression is added to Equation (7.5), to establish a criterion for the longitudinal velocity field in the presence of a density current,

$$u_{max} = |u_{wmax}| + |u_{tmax}| + |u_{smax}| < \frac{\Delta x}{\Delta t} \tag{7.7}$$

The condition of Equation (7.7) leads to a conservative criterion as the worst case conditions for the wind induced circulation and the density current occur in different cells. However, this form of analysis is simpler than a rigorous application of the equation to each cell in the cross-section at the downstream section of each reach.

The conditions for wind induced circulation and density currents in junctions cannot be analysed as above, because the junctions are simply defined as transition zones between adjacent reaches. To determine the effect of junction spatial increments, Equation (7.2) can be used directly using the estimated velocities in cells adjoining the junction. However, this may not always produce stable results because of the coupling of two different numerical schemes, the upwind difference scheme in the junction and the method of second moments in the adjacent reaches, at this point. On a number of occasions, with stability criteria seemingly met, the results become unstable at, and propogated from the cells adjoining a junction of the network. In all cases it was found that halving the time step rectified the problem.

7.2.2 Dispersion Criteria

Written in terms of mass transfer between adjacent cells, the vector form of the dispersion terms of the the three-dimensional mass-transport equation can be written using the dispersion velocities developed in Chapter 5,

$$c\,\vec{u}_D = -\bar{\bar{E}}\nabla c \qquad (7.8)$$

A stability criterion can then be established in the reaches of the system, as dispersion is not modeled in junctions, to state that not more than half the difference in the mass of the substance modeled can be transferred between adjacent cells in the direction of the negative concentration gradient, in one time step Δt. This means that the dispersion coefficient in each coordinate direction has to satisfy the condition,

$$(E_x,\ E_y,\ E_z) \leq \frac{1}{2\Delta t}\ (\Delta x^2,\ \Delta y^2,\ \Delta z^2) \qquad (7.9)$$

Looking at the longitudinal dispersion coefficient, a criterion can be developed from Equations (5.19) and (7.9), for the associated dimensionless dispersion coefficient, K_x,

$$K_x \leq \left(\frac{\Delta x^2}{2\Delta t} - E_o\right) \frac{8.25}{du_{max}} \qquad (7.10)$$

where u_{max} is given from Equation (7.7). For all practical applications, $E_o << \Delta x^2/2\Delta t$, and the condition becomes,

$$K_x \leq 4.125\Delta x^2/\Delta t du_{max} \qquad (7.11)$$

A closer look at the magnitude of the right hand side of Equation (7.11) indicates that this value will usually be much larger than the value of K_x (Taylor's analysis [1954], shows that K_x is on the order of 0.1). Thus for most simulations, this condition will be well satisfied, and can be ignored for all practical purposes.

Similarly from Equations (5.20) and (7.9), the stability criterion for the lateral dimensionless dispersion coefficient, K_y, can be written,

$$K_y \leq 4.125\Delta y^2/\Delta t du_{max} \qquad (7.12)$$

where E_o is again small compared to $\Delta y^2/2\Delta t$. This condition although more readily violated then the condition for the longitudinal dimensionless dispersion coefficient, K_x, is well satisfied in most practical cases as K_y is an order of magnitude less than K_x. The danger arises when many lateral divisions are chosen, and the time step Δt, is large. However, most systems can be reasonably simulated using only one lateral cell in each vertical layer, thus eliminating the need for this condition.

The criterion for the vertical dimensionless dispersion coefficient K_z, is by far the most critical, and usually the one which will be violated first in a simulation. From Equations (5.21), (5.25) and (7.9), the condition can be written,

$$K_z \frac{d^2}{9} \left|\frac{du}{dz}\right|_{max} + E_o \leq \frac{\Delta z^2}{2\Delta t} \qquad (7.13)$$

In this case, E_o can be of the same order of magnitude as $\Delta z^2/2\Delta t$, for reasonable choices of these model parameters. The condition for K_z is then,

$$K_z \leq \left(\frac{\Delta z^2}{2\Delta t} - E_o\right)9/d^2\left|\frac{du}{dz}\right|_{max} \qquad (7.14)$$

where a first condition is that the right hand side of Equation (7.14) must be positive.

Equation (7.14) could be further expanded by introducing the closed form functions of the various components of the velocity field, and differentiating twice to find $\left|\frac{du}{dz}\right|_{max}$. However, this would result in a theoretically valid, but extremely cumbersome, expression to be evaluated at many locations in a canal network. Two simpler expressions can be obtained by considering the conditions, as before, of a wind induced circulation with and without a density current. In both cases the tidal velocity term will be ignored to first order, as its effect is usually small and would automatically be satisfied alone for most practical choices of a network cell structure.

For the first case of the wind induced circulation, Equation (4.30) can be differentiated to given the stability criterion,

$$K_z \leq \left(\frac{\Delta z^2}{2\Delta t} - E_o\right) \frac{18N_z}{d^2K_w|w_s \cos\theta|^2_{max}} \qquad (7.15)$$

This condition uses the value of du/dz at the water surface and hence is conservative.

For the second case in which a saltwater wedge is also present, the condition is much more complicated as it depends on the relative thicknesses of the fresh and saltwater layers. Again, a conservative condition can be established by assuming 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, along with the condition for the wind induced circulation, gives,

$$K_z \leq 9\left(\frac{\Delta z^2}{2\Delta t} - E_o\right) / d^2\left(\frac{K_w|w_s \cos\theta|^2_{max}}{2N_z} + 3u_4\left(\frac{dd_s}{dt}\right)_{max}\left(1 + \frac{d_s}{d-d_s}\right)\right) \qquad (7.16)$$

The criteria expressed in the equations throughout section 7.2, give guidelines for choosing values of the spatial increments Δx, Δy and Δz, and the time increment, Δt. In most cases, the spatial increments will be chosen first, reflecting the amount of detail the user wishes to see in the results. Once these values are established, the time increment can be chosen to meet all the stability criteria. The conditions given in this section act only as a guide to the user. Once the model is operational, an examination of the calculated velocity field will show whether the criteria were met, and simpler forms of the conditions, such as Equation (7.9) directly, can be used for this analysis.

7.3 Convergence Criteria

Let $F(x, y, z, t)$ denote the exact solution of the three-dimensional mass-transport equation in terms of its independent variables x, y, z, and t, and let the exact solution of the difference equation used to approximate it be $\tilde{F}(i\Delta x, j\Delta y, k\Delta z, n\Delta t)$, in terms of its incremental variables Δx, Δy, Δz, and Δt. Then, the numerical scheme is said to be convergent if $\tilde{F} \to F$ as Δx, Δy, Δz, and $\Delta t \to 0$ [Smith, 1975].

In a numerical scheme, there must be a point at which the results are considered to be close enough to the exact solution. Above this point the solution either diverges rapidly from the exact solution, or else the results are not considered to be accurate enough for further use. Below this point, decreasing the incremental variables increases the accuracy of the solution but at the expense of computing time and cost. In other words this point is a point of diminishing return in an economic sense.

For numerical solutions to the three-dimensional mass-transport equation, once the stability conditions are met, the scheme will usually converge to the exact solution. For this equation, with variable coefficients, necessary and sufficient convergence criteria are difficult to establish, particularly with respect to the expected accuracy of the model. In order to establish such a criterion, the model must be run for a variety of values of the incremental variables, and their results compared.

In Table 7.1, a standard data set is given for a 1600 ft long canal with a wind of 5 mph blowing along its longitudinal axis from the tidal entrance to the dead-end (Figure 7.1). Several test cases were run assuming that the initial concentration in the canal was 100 ppm, and that the concentration of the receiving waters was 5 ppm. In all cases the results were depth averaged.

The first test, and the most important, varied the longitudinal spatial increment, Δx, and the time increment, Δt, while using three vertical layers and one lateral layer. For a variety of values of Δx from 50-200 ft and of Δt from 0.097-.388 hrs, the results are shown in Figure 7.2. It can be seen from this figure that the resulting concentration profiles are very similar and predict values that vary by less than 5-6 percent of the initial concentration at any point. This set of results is very encouraging because it includes some combinations of Δx and Δt that are extremely coarse, and which might violate stability conditions for other cases. From Figure 7.2, it can be concluded that the convergence characteristics of the model are such that once the stability criteria are met, the model will yield accurate results.

The second test looked at the number of vertical layers that might be used to simulate a case with wind induced circulation. One lateral layer was defined for simplicity, and the number of vertical layers varied from one to six. It can be seen from Figure 7.3, that the use of only one vertical layer, as expected, averages out the wind effect altogether leaving only a one-dimensional representation of the astronomical tide. The use of two to six vertical layers gave surprisingly similar results for the downstream 2/3 of the canal. However, at the dead-end, the profiles fan out, and for simulations using layers which are not multiples of three, higher concentration values are predicted. It can further be seen that as the number of layers increases from two to five, the concentration profiles at the dead-end tend towards the profiles for the cases of three and six layers. This is not really surprising, because the more layers that are used, the better is the representation of the vertical wind induced velocity profile. Also, even though the depth averaged concentrations are similar, the actual concentrations in each layer differ somewhat from the simulations using three and six layers. The use of many layers to accurately represent the wind induced velocity profile is clearly seen to be an unnecessary waste of time and money, when a simulation using a multiple of three layers will be at least as accurate, and definitely more efficient. For most practical case, three layers are sufficient to model transport conditions, but more can be used if more detailed vertical concentration profiles are required.

The third test was to model the case for three vertical layers, but to vary the number of lateral layers from one to three. As can be seen from Figure 7.4, the results are virtually identical as expected in this case. From this results it may be concluded that one lateral layer is usually sufficient for a simulation. However, two or three layers should be specified in the flow in a bend is considered to be important, or the lateral distribution of concentration is specifically desired. The problem with specifying more than one layer in each cross-section is that the longitudinal distance increment through the junction may be made unreasonably small, as it is the width of the branch canal divided by the number of lateral layers. One method of overcoming this, is to specify a large value for the length of the junction DXJN(*), to give a suitable distance step.

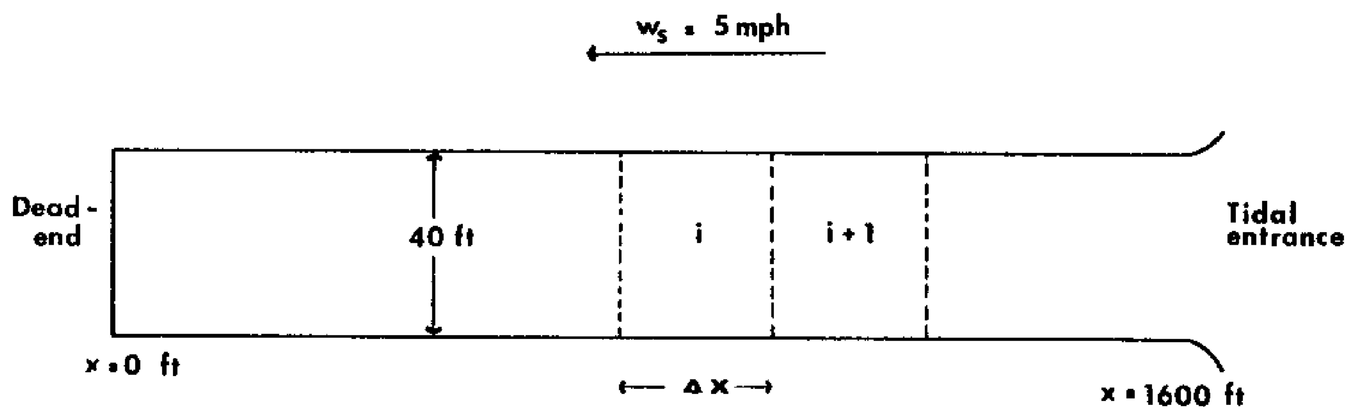


Figure 7.1 - Schematical Canal for Three-Dimensional Model Tests.

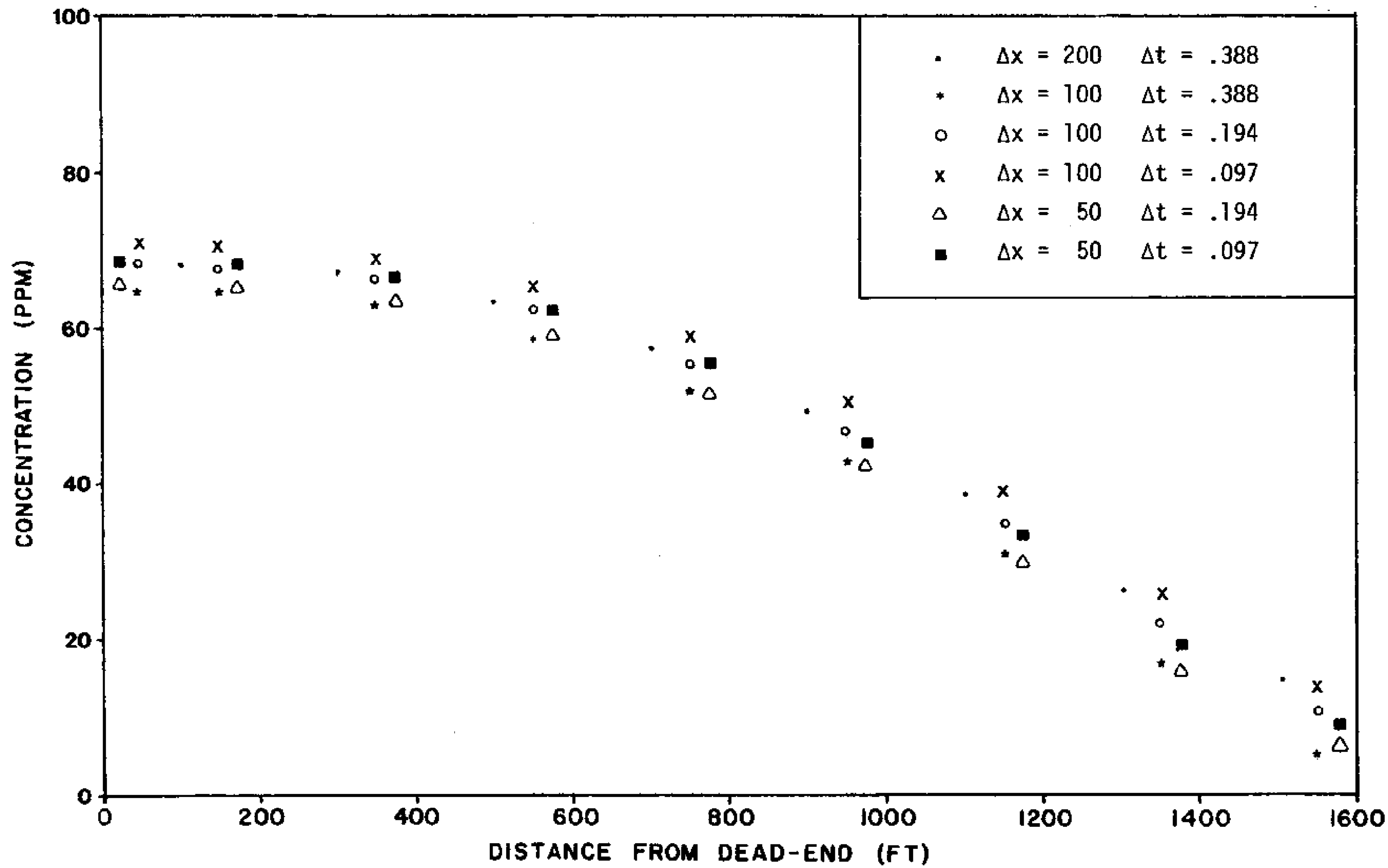


Figure. 7.2 - Variation of Δx and Δt after 50 Tidal Cycles.

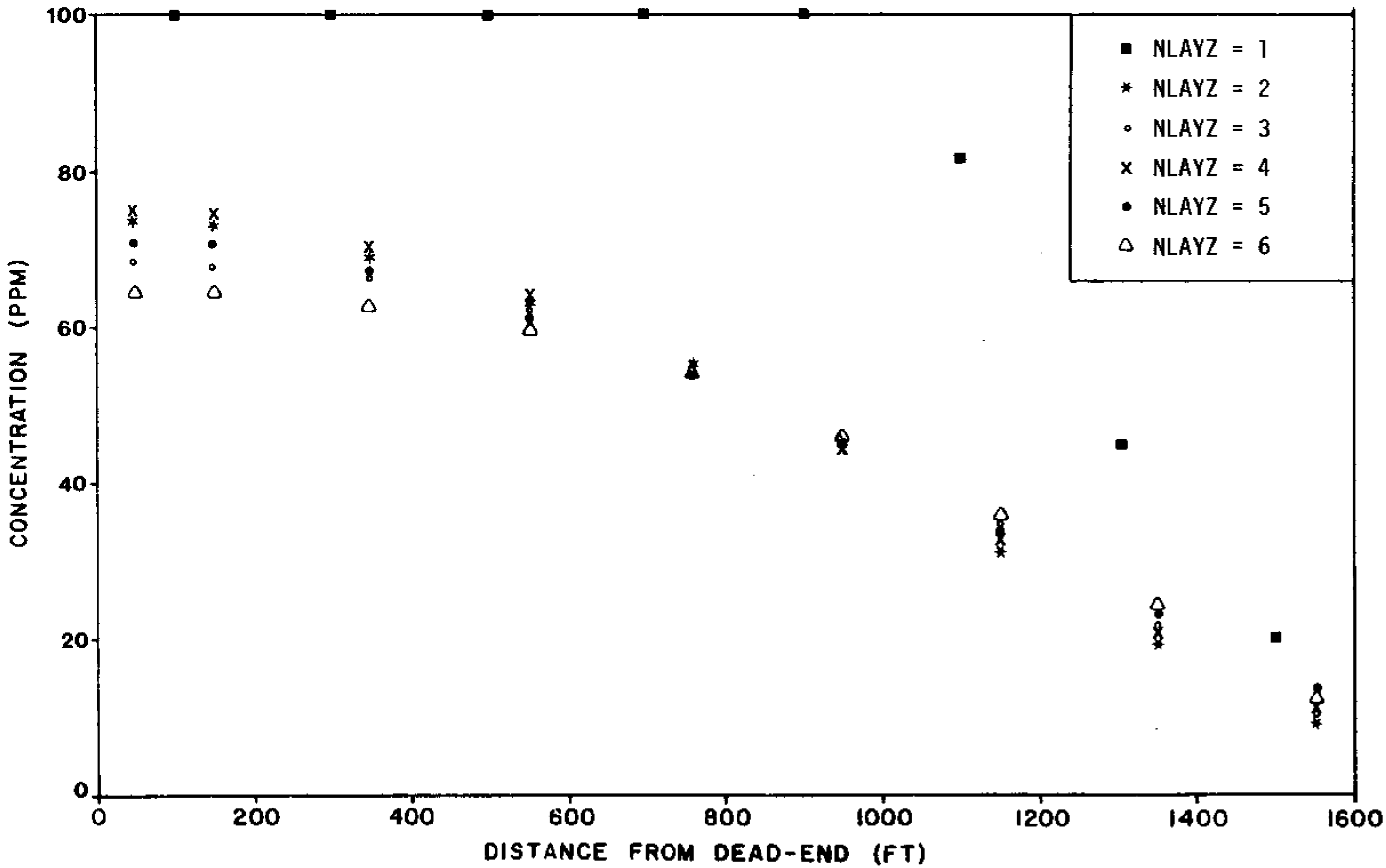


Figure 7.3 - Variations of NLAYZ (NLAYY = 1).

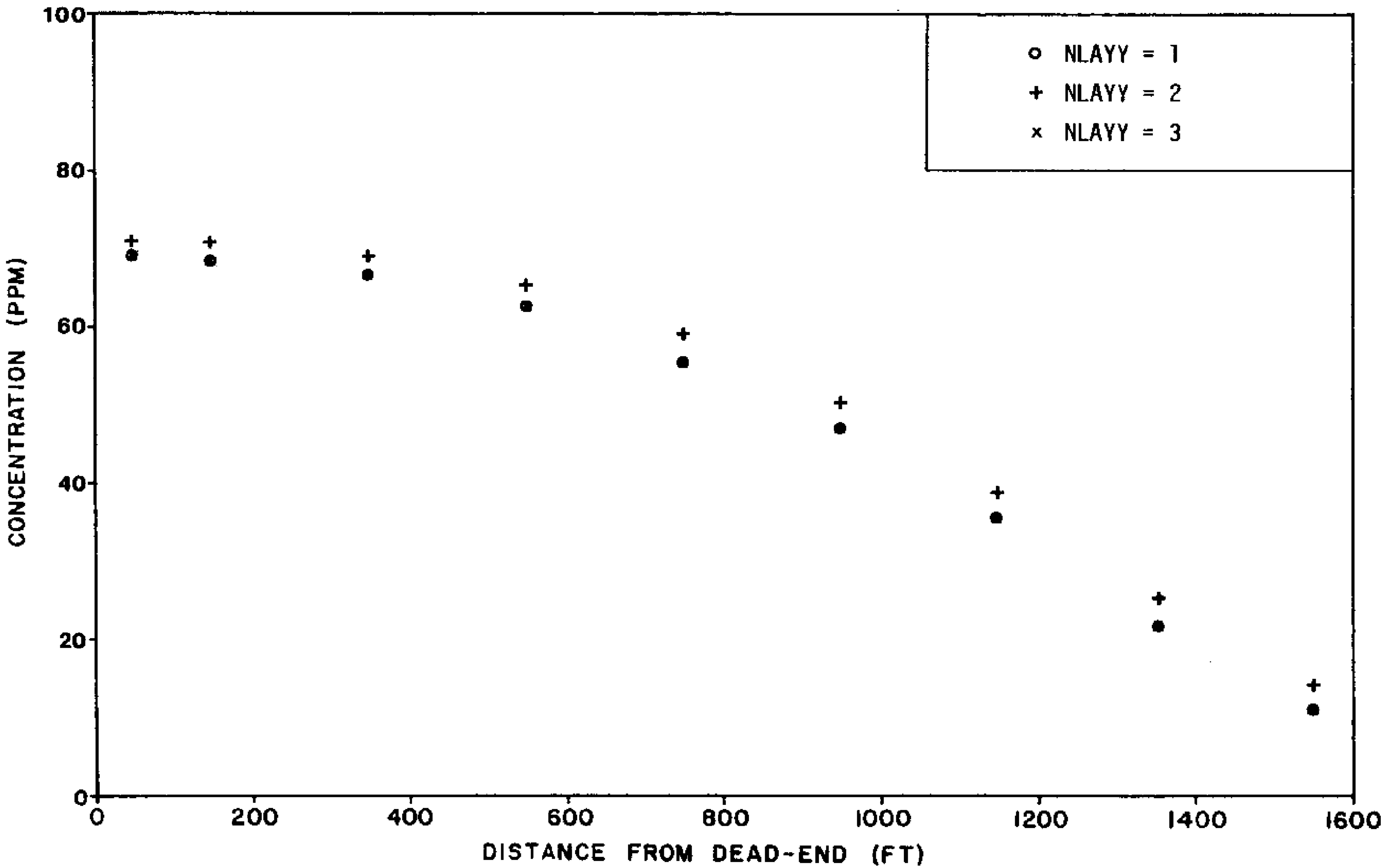


Figure 7.4 - Variation of NLAYY (NLAYZ = 3).

Table 7.1 - 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 mph
Constant wind direction	0 degrees

CHAPTER 8

MODEL TESTS AND RESULTS

8.1 Introduction

In the previous four chapters, a numerical model has been developed to simulate three-dimensional mass-transport in a canal network. The model equations were presented, the organization discussed and a set of user oriented stability and convergence criteria established.

In this chapter several case histories are examined to both calibrate the model and to check its accuracy against field data. The first case examines a study on Big Pine Key, Florida, done by the Environmental Protection Agency [EPA, May, 1975]. Looking in particular at what was called Big Pine Key Canal III, Lee [1977] tested his one-dimensional model against EPA's field data, and arrived at some interesting results. These will be discussed in Section 8.2, and the case will also be analysed from a two-dimensional point of view in an attempt to explain the actual physical processes occurring.

The second case study looks at the measured data collected from the 57 Acres Canal Network, in July and October, 1977. This canal network (presented in Chapter 2) was considered to be influenced only by the astronomical tides and wind induced circulation. During each field investigation, an amount of Rhodamine WT dye was injected near the dead-end of the South Loop (Figure 8.1) at slack tide, and the resulting longitudinal concentration profiles at several succesive slack tides measured using a Turner Designs Field Flurometer. These data were used to calibrate the unknown parameters of the model between adjacent high and low tide longitudinal concentration profiles at the beginning of each study, and were then used to determine whether longer term conditions could be accurately simulated. A discussion is presented in Section 8.3.

The third case, presented in Section 8.4, is the Loxahatchee River study (discussed in Chapter 2), which was considered to be an example of a canal influenced by the progress of a saline wedge into the canal during the flood tide. A field investigation was carried out in June, 1977, and the data used to calibrate the saline wedge parameters associated with the model, and to examine the accuracy of the model in simulating measured conditions through time.

Finally, in Section 8.5, a test canal is set up, and some of the features of the three-dimensional model, which could not be analyzed using the one-dimensional model presented in Chapter 3, are examined. The effect of winds of varying magnitude and directions were modeled for several different cases of lateral inflow loadings. Also, the effect of a saltwater wedge on such a system was studied.

8.2 Case History #1: Results of Big Pine Key Canal III Runs

Big Pine Key Canal III is one of several canals in the Florida Keys studied by the EPA [May, 1975]. The canal (Figure 8.2) 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 NNW.

On November 3, 1973, at high water slack, 500 ml of Rhodamine WT dye was injected at mid-depth, 50 ft from the dead-end of the canal. At regular intervals after the injection, dye concentrations were measured using a fluorometer sampling at mid-depth along the longitudinal centerline of the canal [EPA, May, 1975, Figure 22, p. 45].

Once the data were collected, EPA tried to simulate the profiles using firstly the June, 1973 version of the Storm Water Management Model (SWMM) [Langer et al., 1971, Metcalf and Eddy, et al., 1971], and then the Columbia River Model (CRM) [Callaway, et al., 1969; Callaway and Byram, 1970]. The reason given for turning away from the SWMM model was that:

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

Turning to the CRM, which incorporates a dispersion term, several of the canals examined were studied and the results presented. Although a computer simulation was not shown for Big Pine Key Canal III, one was done for Big Pine Key Canal V, which is an identical, adjacent canal to the west. Using a measured longitudinal dispersion coefficient of 24×10^{-4} sq mile/day (0.77 ft^2/sec) at the mid-point of the canal as a uniform coefficient, they managed to reproduce the measured profiles by making an adjustment to the dispersion term, effectively by <u>dividing</u> it by five.

Interestingly, Lee [1977] also tried to model Big Pine Key Canal III using a one-dimensional model which corrects for numerical dispersion (described in Chapter 3), with a linearly varying longitudinal dispersion coefficient that matched the measured value of 26×10^{-4} sq mile/day (0.84 ft^2/sec) at mid-canal. However, in obtaining a fit between computed and observed profiles he had to <u>increase</u> the dispersion term by <u>two</u> orders of magnitude (Figure 8.3).

From these results, two conclusions may be drawn. Firstly, the CRM suffers from severe numerical dispersion when used to model low energy canals such as those along the Gulf and East Atlantic coasts. This is not really too surprising, as the CRM and SWMM are interrelated models, both having been developed from the Dynamic Estuary Model (DEM) [Feigner and Harris, 1970]. The inclusion of a dispersive term in the CRM cannot be expected to produce any better results, as the main source of error has not been addressed.

Secondly, as stated in the EPA report [May, 1975], "the dye cloud centroid remaining in one place" must be expected in a model which, due to its one-dimensional form, can only model the velocity field induced by the astronomical tide. In the Big Pine Key area of Florida, the tidal range is on the order of 1 ft. A simple volumetric tidal prism analysis indicates that maximum velocities of no greater than 0.01 ft/sec can be expected, and then only at the tidal entrance. At the dead-end of the canal, the tide induced velocities will be much lower, and certainly not enough to move the dye cloud any appreciable distance.

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

To illustrate this point, the three-dimesional model, CANNET3D, was run for this case. One lateral layer and three vertical layers were used and the wind speed and direction estimated from monthly averages for the Key West area (Table 8.1). From these data, a resultant wind in the canal appeared to be about 5 mph blowing from the tidal entrance to the dead-end. The remaining coefficients left to vary were the vertical momentum transfer coefficient, N_z, the background dispersion coefficient, E_o, and the three dimensionless dispersion coefficients, K_x, K_y, and K_z. E_o was set to zero as the wind speed was constant, and thus its effect is combined with that of K_z. As there was only one lateral layer, K_y has no effect. Also, the effect of K_x is very small, and the two important parameters are N_z and K_z.

To calibrate these parameters, 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 at the same time matched. It can be seen from Figure 8.3 that good agreement was found by setting N_z = 0.005 ft^2/sec, and K_z = 0.008 ft^2/sec (Table 8.1). As will be seen in the next section, these are very reasonable values for N_z and K_z particularly considering that the variability of the wind could not be introduced into the model, as wind data were not collected at the site, and also that an estimate was made, however realistic, of the magnitude of the wind. As the wind induced velocity depends on the wind speed raised to the power 2.5 (Equations (4.18) and (4.27)), N_z could easily have the same value as found for the 57 Acres Canal Network, discussed next. Furthermore, decay was not introduced because the time sequence and atmospheric conditions at the time of the test were unknown.

In conclusion, it has been demonstrated quite clearly, that a one-dimensional model is quite useless in predicting water quality conditions in low energy canal systems. In many cases, the tidally induced flow is one of the least significant forcing functions, and thus field conditions cannot be hoped to be simulated using only one component of the physical phenomena at work. In a simple testing of the three-dimensional model, using only a two-dimensional capability, a much more realistic set of results were obtained, using values of the variable parameters that will also be seen to be applicable to other canal networks in different locations and operating under different conditions.

8.3 Case History #2: Results of 57 Acres Model Runs

The 57 Acres Canal site has been extensively investigated by the Hydraulic Laboratory since early 1975. The progression of research on this site was discussed at some length in Chapter 2. However, two particular field studies have been isolated for this analysis because each contains a detailed study of the movement of a dye cloud introduced into the system.

During both the July and October field trips in 1977, 1500 ml of Rhodamine WT were released into the end section of the South Loop of the system (Figure 8.1) at slack tide. For the July study the dye was released as a point source, while in October it was mixed throughout the width, over the top 1/3 of the depth, using the pressurized release technique discussed in Chapter 2. At each successive high and low tide, a longitudinal concentration profile was measured at a depth of 3 ft below the water surface, using a Turner Designs Field Fluorometer. The sampling was done by driving the boat at low speed down the center-line of each reach of the system.

A wind gauge was installed for the October Field Survey so that a continuous record of wind speed and direction could be maintained throughout the course of the investigation. To investigate the results of the July study, a wind record for the closest permanent wind gauge at West Palm Beach Airport, Florida, was obtained. These two records showed quite different wind conditions. From Figure 2.35, it can be seen that the wind during the October investigation was fairly light, becoming very calm at night, whereas for the July study, the wind was much stronger and more uniform with 10-13 mph winds out of the East for much of the time. During both investigations, a continuous recording tide gauge was set up in the dead-end of the South Loop to obtain a time history of tidal elevations which, like the wind speed and direction, was digitized for input to the model.

For both these test cases, it was decided to simulate mass transport in the South Loop only, for computational economy, and because not much of the dye escaped from this area during the 40-50 hrs of simulation in each case. Thus the entrance to the South Loop was set up as a tidal entrance, and a high time decay rate, τ = 6.21 hrs (half the tidal period) was specified because of the small receiving waterbody present.

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 8.1), was divided into 100 ft segments, and one lateral layer and three vertical layers were chosen for the model. Therefore, it was assumed that initially, the dye was well mixed throughout the upper layer of the third segment from the dead-end (200-300 ft). This gives an initial concentration in that cell of,

$$c_i = \frac{(1500 \text{ ml})\ (3.531 \times 10^{-5} \text{cu ft/ml})}{(3 \text{ ft})\ (100 \text{ ft})\ (100 \text{ ft})} \times 10^9 \text{ ppb}$$

$$= 1750 \text{ ppb} \qquad (8.1)$$

The background concentration, c_{RW}, was estimated to be 20 ppb from field observations. The observed and computed longitudinal concentration profiles for reach numbers 1 and 5, and junction number 2 are shown in Figure 8.4.

During the October study, a similar amount of dye was injected into the canal a little further away from the dead-end. For this study case, it was decided to digitize the longitudinal concentration profile in reach number 1 at the first high water slack tide after the injection. Again from field observations, the background concentration was found to be 0.2 ppb. The observed and computed longitudinal concentration profiles for reach numbers 1 and 5, and junction number 2, for the October study, are shown in Figure 8.5.

As for the Big Pine Key Canal III study case, the important parameters to investigate were the momentum transfer coefficient, N_z, and the dimensionless vertical dispersion coefficient, K_z. However, in this case, as the wind conditions during each investigation were different, the effect of the background dispersion coefficient, E_o, could also be studied.

For both the July and October simulations, the first runs in each case were designed to vary N_z until the model predicted the observed location of the center of mass after half a tidal cycle (or to the following slack tide). Both runs met with some success and some difficulty. For the July run, the fact that wind data was from an off-site source, was reflected in the fact that it was difficult to accurately simulate actual conditions. However, it can be seen from Figure 8.4, that the general movement of the dye cloud is fairly well followed.

The results for the October simulation, (Figure 8.5), indicate that the center of mass is followed fairly well after the first slack tide from the beginning of the simulation. However, it is not fully understood why the initial fit to the profile after one-half tidal cycle is not better. During this period from 10 p.m. on October 18, 1977 until 4 a.m. on October 19, it can be seen from Figure 2.35, that there was no wind blowing, and yet the dye cloud moved downstream against a flood tide. The probable explanation of this apparent inconsistency is data error.

For both cases it was found that a value of N_z = 0.002 ft^2/sec gave fairly good agreement between the simulated and observed locations of the center of mass of the dye cloud throughout the period of investigation. This result agrees with the value obtained from the Big Pine Key Canal III study case (Section 8.2), when it is recalled that an estimated wind velocity was used. In fact, as the wind induced velocity in the canal is related to the wind speed to the power 5/2 for moderate winds (Equations (4.18) and (4.27), the wind speed in the latter case would only have to be multiplied by a factor of 0.7 to give exact agreement between the two simulations.

Once the transient locations of the center of mass of the dye clouds had been simulated, the next step was to reproduce the peak values of the dye concentration profile. First runs indicated that using K_z alone would not match both sets of field data. During the initial period of the October investigation, the winds were either very light or dead calm. Thus, it was decided to use the background dispersion coefficient, E_0, to match the peak concentration values for the first measured profile after the start of the simulation. When this was done, E_0 was held constant and K_z adjusted to match the peak concentration values for the first measured profile after the start of the July simulation. This formed the basis of an iterative procedure on the values of E_0 and K_z until good agreement over the initial period of the two simulations was found. The values obtained were E_0 = 0.0005 ft^2/sec and K_z = 0.0001.

With these values, and others given in Table 8.2, the simulations were then run to the end of the periods of record. The results are plotted in Figure 8.4 for the July investigation and in Figure 8.5 for the October investigation. From these figures, it can be seen that reasonably good agreement was found considering the difficulties inherent in obtaining accurate data over a long time period. Most of the disagreement is believed to have been caused by the continual recalibration of the fluorometer during each study. The results show that the mass of dye present in the canal varies during the measurement period in a manner not consistent with decay, mixing to lower layers, and loss through the tidal entrance. Considering this to be the case, the degree of fit between model and prototype values is good.

In the South Loop at Station 3 (Figure 2.27), there is a sharp bend. For the simulations described above this feature was not included because of the variability of measured concentration, and because the concentrations were only measured at a depth of 3 ft. During the October field investigation, three velocity towers were set up just downstream of the bend, and a period of record obtained. To investigate the accuracy of the model, this bend was included and the model run for the October study case with three lateral layers and three vertical layers. A predicted and measured lateral velocity field were shown in Figure 4.7, where the theory of secondary currents was discussed. From this figure it can be seen that in spite of the assumptions made, field conditions are well modeled.

During this same run, the vertical dispersion coefficients, E_z, were also printed out, and are shown in comparison with the component of wind down reach number 1 (Figure 8.1) in Figure 8.6. From this figure, the importance of modeling this parameter based on the velocity gradient can be seen. Many modelers simply specify E_z to be constant in their models. However, this can be unrealistic in low energy tidal canals in which flow reversals are common. In the reversal layer, a good deal of 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 this point.

In conclusion of this section, good agreement was found between the various studies for the value of the vertical momentum transfer coefficient, $N_z = 0.002\ ft^2/\ sec$. The results of simulating the July and October dye studies yielded $E_o = 0.0005\ ft^2/sec$ and $K_z = 0.0001$. These values gave good results considering the difficulties involved in obtaining accurate field measurements.

A sample test run for the October simulation is shown in Appendix G. The input parameters are listed, the results printed out after every 5 hrs of simulation, and longitudinal concentration profiles plotted at two time intervals during the simulation.

8.4 Case History #3: Results of Loxahatchee River Runs

The Loxahatchee River site (Figure 2.11) 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 (Figure 2.12). The alignment angle of the canals is approximately 270 degrees (west). During a field investigation by the Hydraulic Laboratory in June, 1977 (discussed in Chapter 2), 75 ml of Rhodamine WT dye was injected and mixed in the upper half depth, across the width, 200 ft from the dead-end of the North Canal at high water slack. Sampling was performed at regular intervals at discrete points in the vertical and along the longitudinal centerline of the canal, as indicated by the readings in Figure 2.25.

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 permanent 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 2.23) 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 Chapters 4 and 6, to model a saline wedge requires that the simulation begins at low tide. Unfortunately, the dye was injected into the system 3 hrs before low tide. However, the first set of measurements were taken during the following low water slack tide, and 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 hrs. This test run was used to study the salt wedge parameter, u_4, introduced in Equation (4.64). 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 8.3.

From Figure 8.7, it can be seen that good agreement was found between observed and computed concentration values for $u_4 = 3000$. Also, giving confidence to this value, the computed vertical velocity profile as shown in Figure 4.10, matches well the observed profile measured at the corresponding time.

As stated before, only one canal exhibiting these 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 system.

8.5 Effect of Varying Model Parameters on Three-Dimensional Mass-Transport

In Section 3.8 the effects of the different parameters of the one-dimensional model were studied and some conclusions drawn. The main parameters varied were geometric, and thus their qualitative effect as presented in the variability analysis will also be the same for the three-dimensional model. However, the one-dimensional model does not have the capability of modeling phenomena which produce zero net flux through any cross-section of a reach. These phenomena, such as wind induced circulation and density currents, can only be studied by a two- or three-dimensional model, and will be the subject of the following two subsections.

The following analysis was based on a single prismatic canal whose dimensions are those of the Big Pine Key Canal III discussed in Section 8.2 (Figure 8.1), but whose longitudinal axis is directed south (180 degrees). The other parameters were chosen fairly arbitrarily, being selected to provide a constant data set rather than reflect actual conditions in that particular canal. This data set is listed in Table 8.4.

The analysis of the following two subsections serves to provide the reader with a qualitative idea of the effect of wind induced circulation and density currents on a simple system. A much more in-depth look at these effects will be presented in later chapters, and in a separate dissertation [Morris, 1978]. In those discussions, the entire canal network design process will be studied, and a procedure outlined for investigating proposed systems with a view to optimizing their flushing characteristics.

8.5.1 Effect of Wind Induced Circulation

In this section, the effect of wind induced circulation will be studied in the absence of a density current. For each case considered, five different wind conditions were used which are, zero wind (0,0), 5 mph blowing out of the canal (5,0) (positive x-direction), 5 mph blowing into the canal (5,180) (negative x-direction), 10 mph blowing out of the canal, (10,0), and 10 mph blowing into the canal (10,180). In the following figures, these conditions will be labeled on the plots as "(wind speed, wind direction)".

The first case, case 1W (see Table 8.5, where W refers to wind induced circulation only), examined the rated of flushing of the system due to wind. The initial concentrations in the canal, c_i, were set to 100 ppm and the background concentrations, c_{RW}, was 5 ppm. Such might be the case of a catastrophic pollutant loading on the canal. The resulting vertically integrated longitudinal concentration profiles after ten and fifty tidal cycles are shown in Figure 8.8 and 8.9 respectively.

As expected, the resulting profile with no wind shows that very little of the substance has been removed from the canal. In fact it would take many hundreds of tidal cycles to cause any significant effect on the system. This result is in complete accord with results obtained for a similar system using the one-dimensional model, and clearly demonstrates a one-dimensional model's inability to accurately model the flushing characteristics of low energy tidal canals.

From these figures it can be seen that the 10 mph wind has flushed half the substance from the canal in ten tidal cycles, and virtually all of it after fifty tidal cycles. The results for the 5 mph wind are much more interesting. Examining Figure 8.9, it can be seen that with the wind blowing out of the canal, over 50 percent of the substance was removed after fifty tidal cycles, whereas with the wind directed into the canal, about 10 percent more has been retained in the same time. This is due to the fact that transport out of the canal in the lower layers for this wind condition is slower, because of friction at the bed, than for transport out in the upper layer with an out-blowing wind.

This can also be seen in the second test case, case 2W (Table 8.5), in which the two concentrations are reversed. The initial concentrations in the canal were 5 ppm and the background concentration of the receiving waters set at 100 ppm. This might be an example of a "clean" canal connected to a polluted waterbody.

The results after ten and fifty tidal cycles are shown in Figures 8.10 and 8.11 respectively. From these figures it can be seen that the exact opposite of the previous case is found. For zero wind, hardly any intrusion is found, which would be an underestimation in any practical situation. For wind speeds of 10 mph, the concentrations in the canal virtually increase to the background after fifty tidal cycles, and as can be seen for the case of the 5 mph winds, the higher concentrations result when the wind is blowing into the canal from the tidal entrance towards the dead-end.

The next series of runs examines the effect of wind induced circulation for the case in which there is a uniform lateral inflow of a substance with concentration c_I = 100 ppm, distributed in various ways along the length of the canal. In each case, the total volume rate of the substance entering the canal per unit time is constant.

In the first simulation, case 3W (Table 8.5), a uniform lateral inflow rate of q_I = 0.04 cu ft/hr/ft, with a concentration c_I = 100 ppm was specified along the length of the canal. The resulting longitudinal concentration profiles after ten and fifty tidal cycles are shown in Figures 8.12 and 8.13 respectively. Here it can be seen that for the case of no wind, the concentration at the dead-end is steadily increasing. Also, the wind blowing out of the canal is much more effective in removing the substance than for a wind blowing into the canal. The result is not too surprising, as a wind blowing into the canal transports the substance towards the dead-end in the upper layers, and then towards the tidal entrance in the lower 2/3 of the depth. Thus the substance has further to go to reach the tidal entrance and takes much longer to do so than for either case of the wind blowing out of the canal. In fact, for a 10 mph wind blowing out of the canal in this case, no measurable amount of the substance above the background value remains in the canal, and for a 5 mph wind, the increase in concentrations throughout the length of the canal is small.

The second distribution, case 4W (Table 8.5), results from an inflow rate q_I = 0.08 cu ft/ hr/ft in the upper half of the canal. The resulting concentration profiles after ten and fifty tidal cycles are shown in Figures 8.14 and 8.15 respectively. The build-up at the dead-end for no wind or for a wind blowing into the canal, is now more pronounced. As before, a 10 mph wind blowing out of the canal succeeds in keeping pace with the inflow rate in this case, and the increase in concentrations is small.

The 5 mph wind blowing out of the canal is not as efficient as before because more of the substance is now diffused into the lower layers as the concentration difference is larger than before, and carried back towards the dead-end. For a 5 mph wind blowing into the canal, the concentration is building up at the dead-end, as the induced velocities in the lower layers are not sufficient to carry out all the inflow. An equilibrium position will be reached with time. However, the 10 mph wind in this case induces a sufficiently large return velocity to cause the concentrations at the dead-end to increase very slowly through time.

The final simulation, case 5W (Table 8.5), considers a large loading at the dead-end of the canal. In this case, the lateral inflow rate is 0.32 cu ft/hr/ft along 200 ft adjacent to the dead-end. The resulting profiles after ten and fifty tidal cycles are shown in Figure 8.16 and 8.17 respectively. In all cases, the wind induced circulation was unable to deal with the continuous loading. However as before, concentration values were lower with the wind blowing out of the canal than blowing into the canal for each wind speed.

In case 5W, however, the 5 mph wind blowing out of the canal was not as effective in reducing concentration values as the 10 mph wind blowing into the canal. This shows that there is some point in the variability analysis at which the transfer of substance vertically becomes important when compared to purely convective transport of the velocity field. In this case, the 5 mph wind blowing out of the canal, is not able to transport the substance out of the canal quickly enough to prevent a considerable amount of vertical transfer to the lower layers which is transported towards the dead-end.

In conclusion, most cases of lateral inflows of pollutants into canals, occur into the top layers of the canal; such as run-off from lawns, leaking septic tanks, etc. In these situations, the above analysis shows that the resulting pollutant mass is better flushed when the wind is blowing down the length of the canal from the dead-end towards the tidal entrance, and that peak concentrations in the canal are smaller than for winds of equal magnitude blowing into the canal.

Also, in cases in which the receiving waterbody has higher concentrations than those desired in the canal, a wind blowing out of the canal is more effective in flushing whatever pollutant does enter the system. Again maximum concentration values in the canal are smaller in this case than for a wind blowing into the canal.

However, if the main pollutant source is in the lower layers, and is located away from the dead-end where it might quickly be transferred to the top layer and transported out of the canal, then a wind blowing into the canal might be more desirable to obtain optimum flushing.

8.5.2 Effect of Density Current

The previous section described the flushing characteristics of the test canal under the influence of wind induced circulation alone. In this section, a density induced circulation is superimposed on the same system and the results analysed for the test cases 1S - 5S listed in Table 8.5. For this discussion, only two wind conditions were used, a 5 mph wind blowing into the canal, and a 5 mph wind blowing out of the canal.

The salt wedge interface used in these test runs was located 5 ft below the mean tidal elevation at the tidal entrance to the canal, and the coefficient, u_4, of Equation (4.64) was assigned the value 4000.

The simulations were carried out to fifty tidal cycles and the results from this analysis presented only at the completion of each run.

For the first test case, case 1S, in which the initial concentrations in the canal were 100 ppm, and the background concentration was 5 ppm, the results are shown in Figure 8.18. Because of the greater circulation induced by the movement of the salt wedge, sufficient to overcome the smaller wind circulation in this case, the resulting longitudinal concentration profiles are significantly lower. As before, the resulting profiles with the wind blowing out of the canal are lower than for the wind blowing into the canal, because the wind induced reversal in the lower layer aids the passage of the salt wedge into the canal and much more mixing occurs across the interface. Conversely, in the case in which the wind blows into the canal, the induced flow reversal tends to impede the density current reducing velocities and thus decreasing vertical dispersion.

In case 2S, in which the initial concentrations in the canal are 5 ppm, and the background concentration of the receiving waterbody is 100 ppm, the opposite is true (Figure 8.19). As the passage of the salt wedge dominates the wind induced circulation for these wind speeds, much more of the higher concentration receiving waterbody is circulated in the canal giving higher concentrations than before. In this case, the 5 mph wind blowing out of the canal does not serve to retard this effect, but instead increases it by aiding the passage of the wedge with a flow reversal towards the dead-end in the lower layers. Also, larger vertical velocity gradients are present which increase vertical dispersion.

In cases 3S to 5S, which examine various lateral inflow distributions, q_I, of constant concentration, c_I = 100 ppm (Table 8.5), the results show that in each case (except case 3S with the wind blowing out of the canal) the resulting longitudinal concentration profiles are similar to the cases of no density current, but that the effect of the density induced circulation is to improve mixing and lower the range of values of the profiles (Figures 8.20 - 8.22). The discrepancy in the case 3S result is probably due to some small instability present in the salt wedge computations in which two different schemes are matched. The wedge induced circulation increases the flow out of the canal in the upper layer of the flow, and also increases the vertical dispersion to the lower layers. As the wedge retreats during the ebb tide, the mass of substance in the lower layers is carried out to the receiving waters.

From these results, it would appear that the effect of the movement of a saltwater wedge in a freshwater canal is to improve the circulation in that canal, and flush out substances carried by both the fresh and saltwaters. However, the results also show that if a substance is being introduced from the receiving waterbody, such as saline water itself, the transition to a similar type condition in the canal will be quicker than for the case of wind induced circulation alone.

The results in this and the previous subsection are included to provide some qualitative idea of the effect of different external phenomena on water circulation and the flushing characteristics of tidal canals. They can serve as a guide only because there are so many combinations of conditions that can occur, that an all-inclusive analysis is not possible. Each case must be examined on its own mertis. For example, in Floridian canals, the above analysis might indicate that the West Coast (or Gulf Coast) canals have better flushing characteristics than East Coast (or Atlantic Coast) canals when they are built off tidal estuaries with large freshwater river inflows. However, the effect of the salt wedge movement in such a comparison must be weighed against the usually small tidal ranges in this region.

Thus, a proposed canal network should be analysed comparing different conditions which might be expected at a site in that location. Accurate wind and tidal elevation records should be obtained in the area, and a comparative analysis performed for that site with variations in canal network layout. The techniques used in such an analysis are outlined in later chapters of this report, and a design case studied.

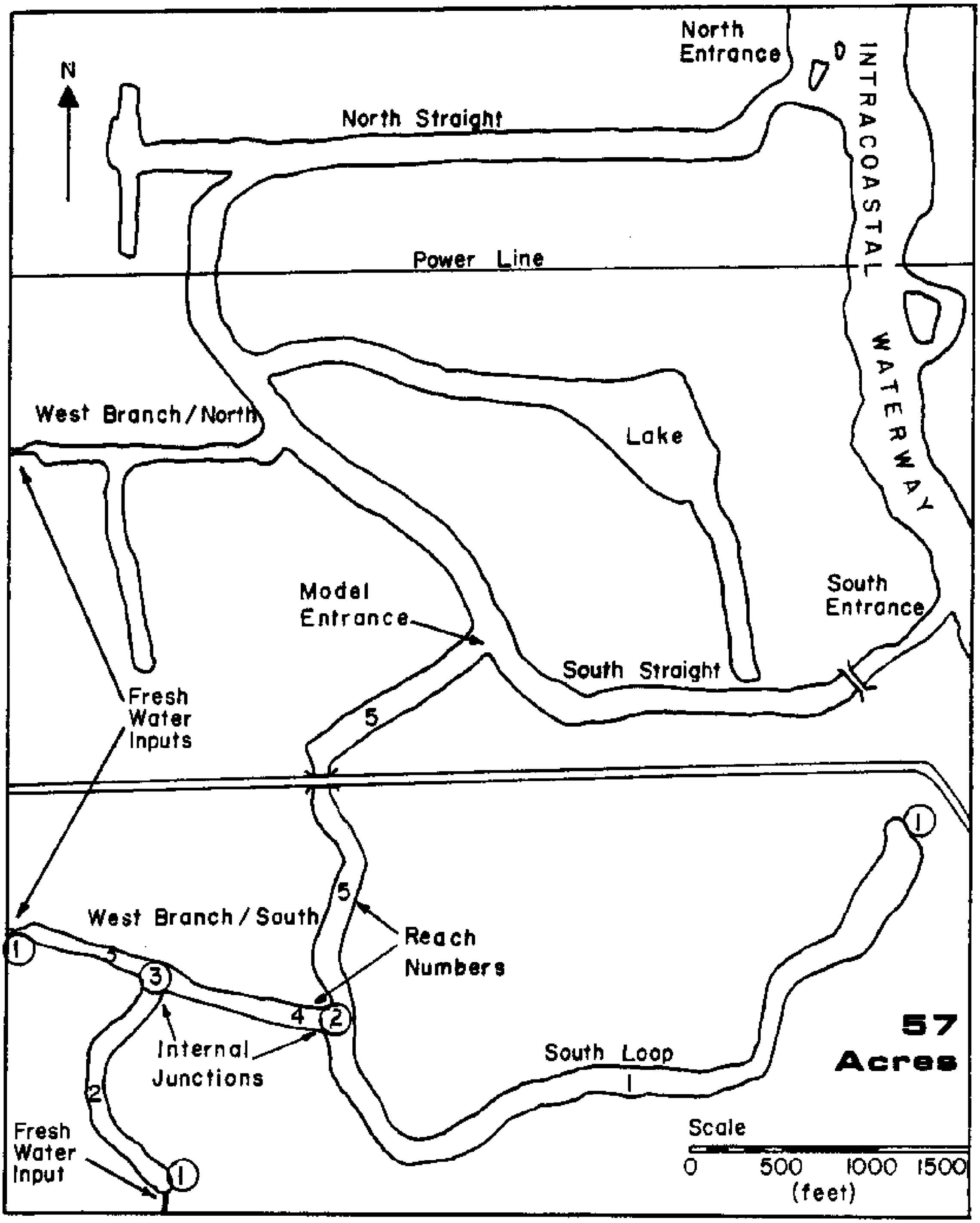


Figure 8.1 - Model Layout of Reaches and Junctions in 57 Acres System.

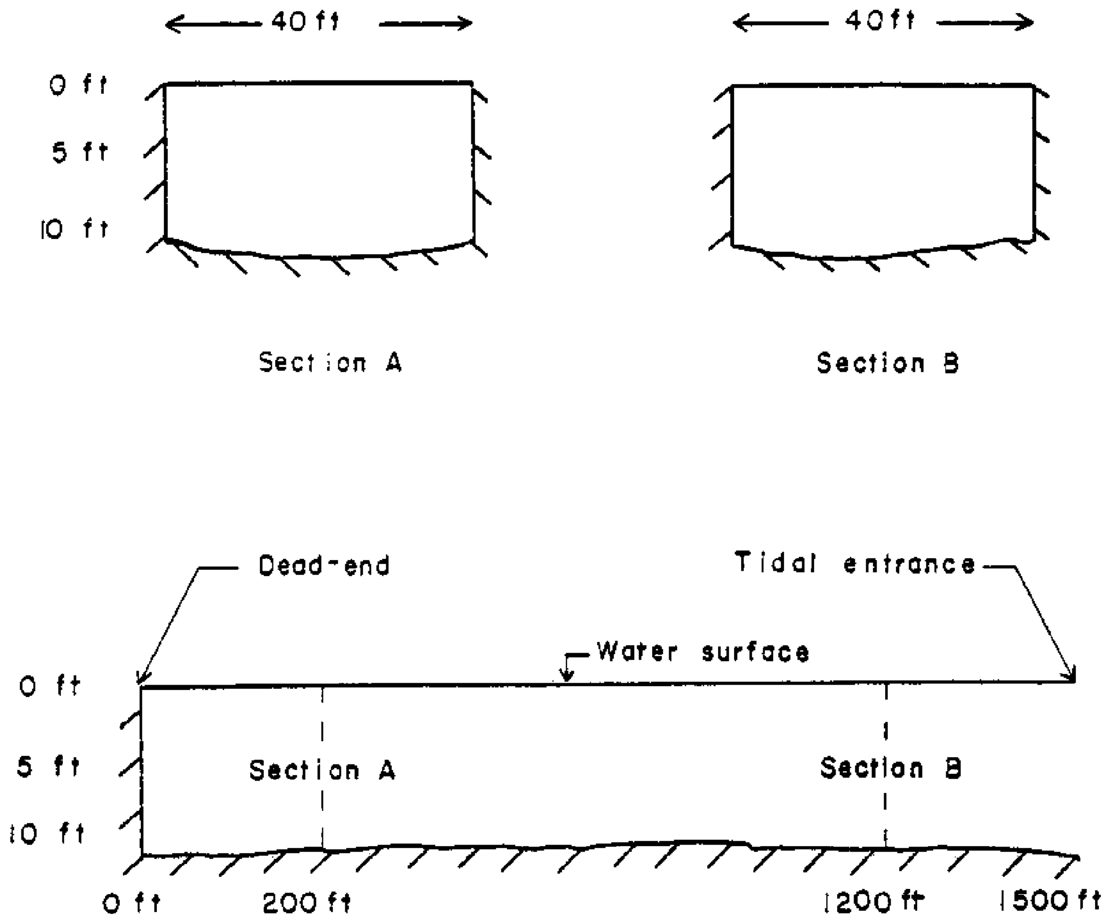


Figure 8.2 - Typical Longitudinal Sections and Cross-Sections, Big Pine Key Canal III, Florida [EPA, May, 1975].

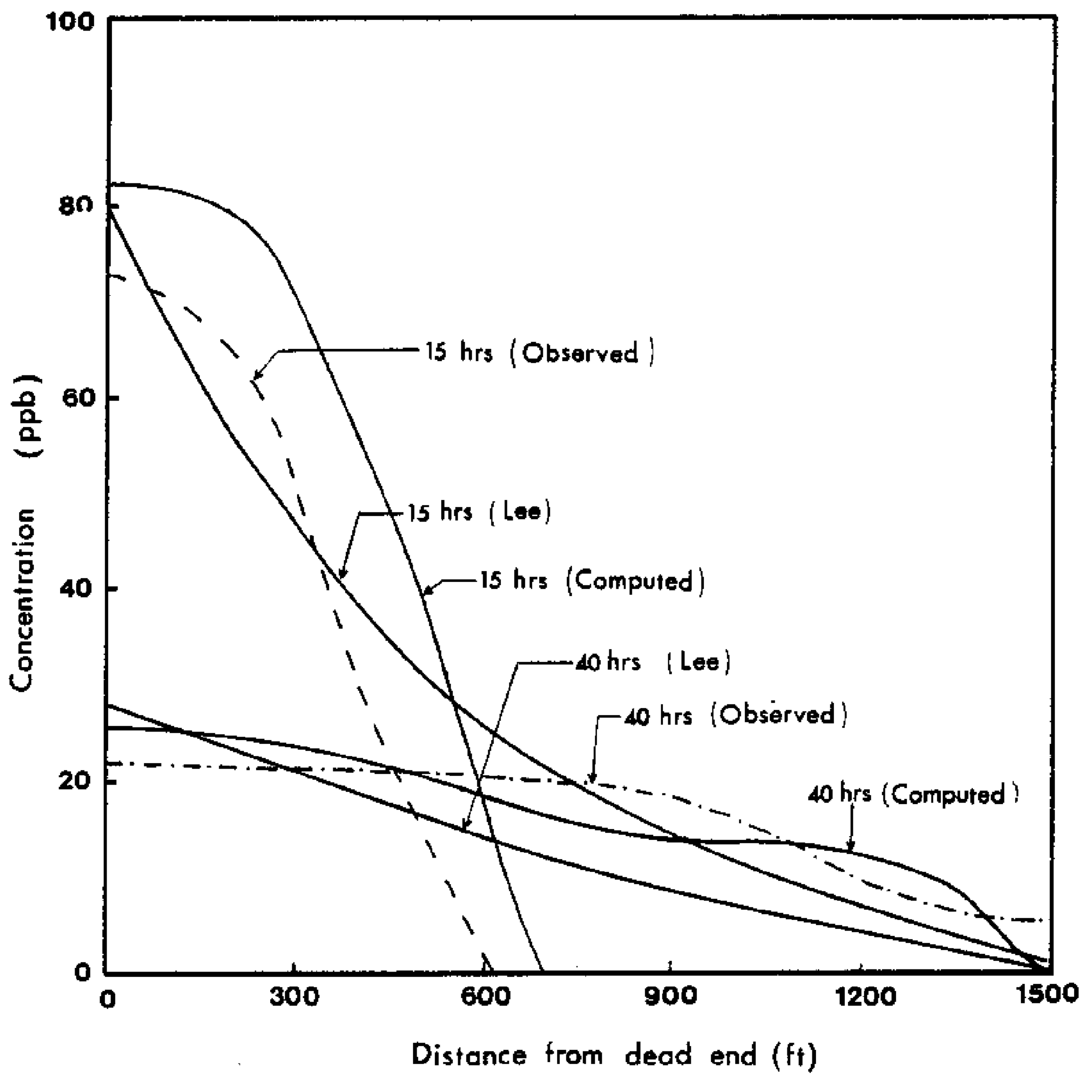


Figure 8.3 - Observed and Predicted Concentration Profiles for Big Pine Key Canal III Case History.

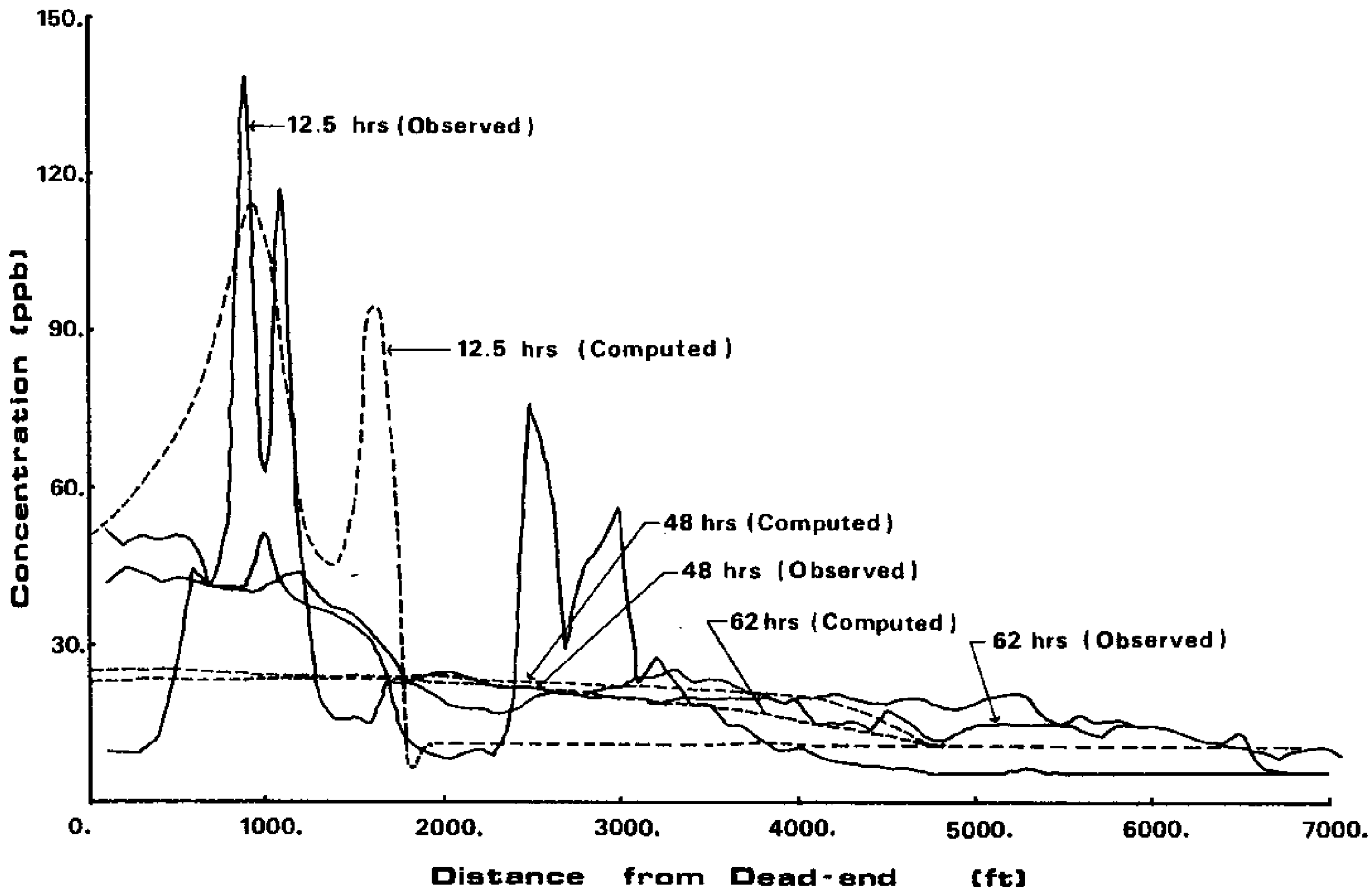


Figure 8.4 - Observed and Prediced Concentration Profiles for 57 Acres July, 1977 Case History.

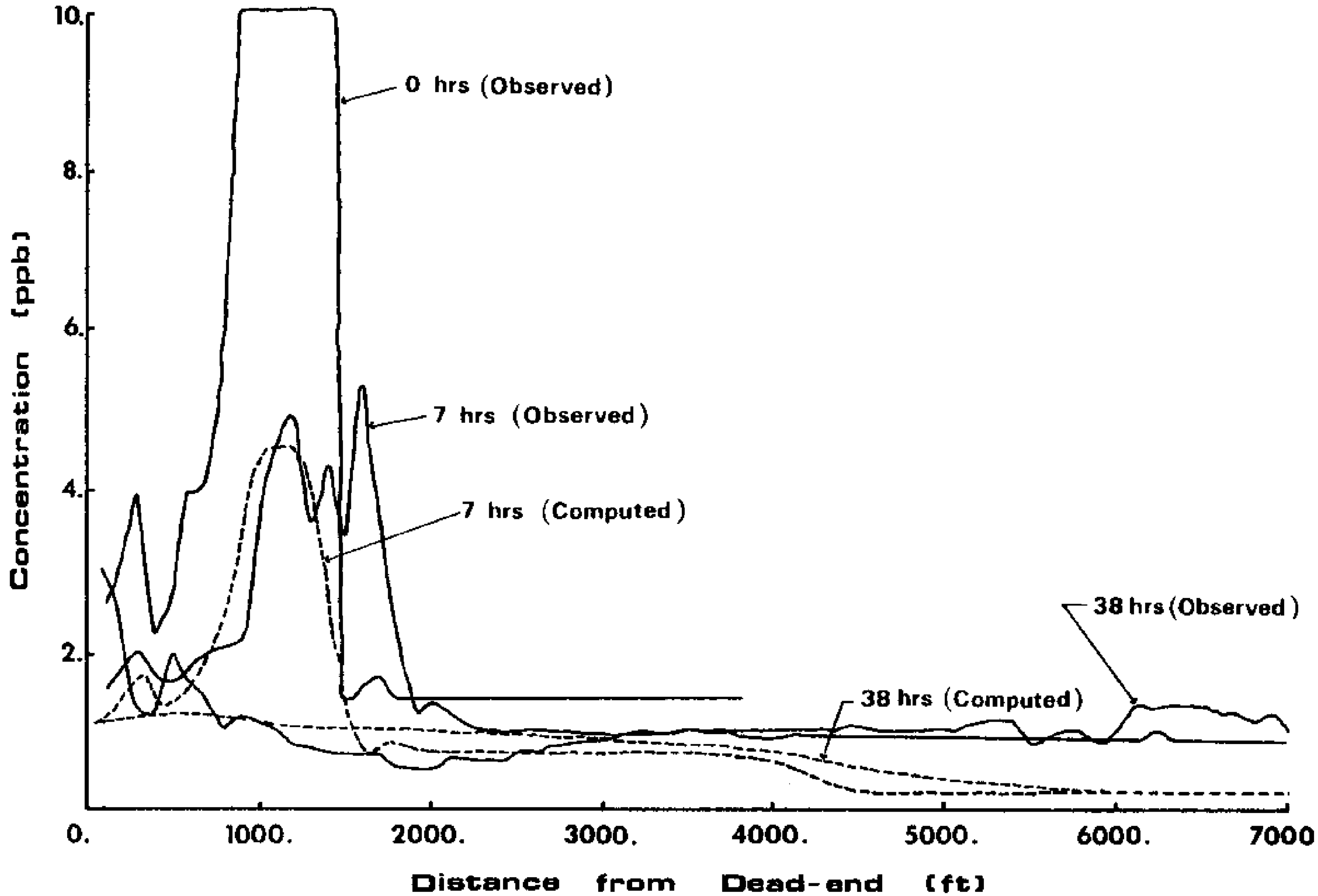


Figure 8.5 - Observed and Predicted Concentration Profiles for 57 Acres October, 1977 Case History.

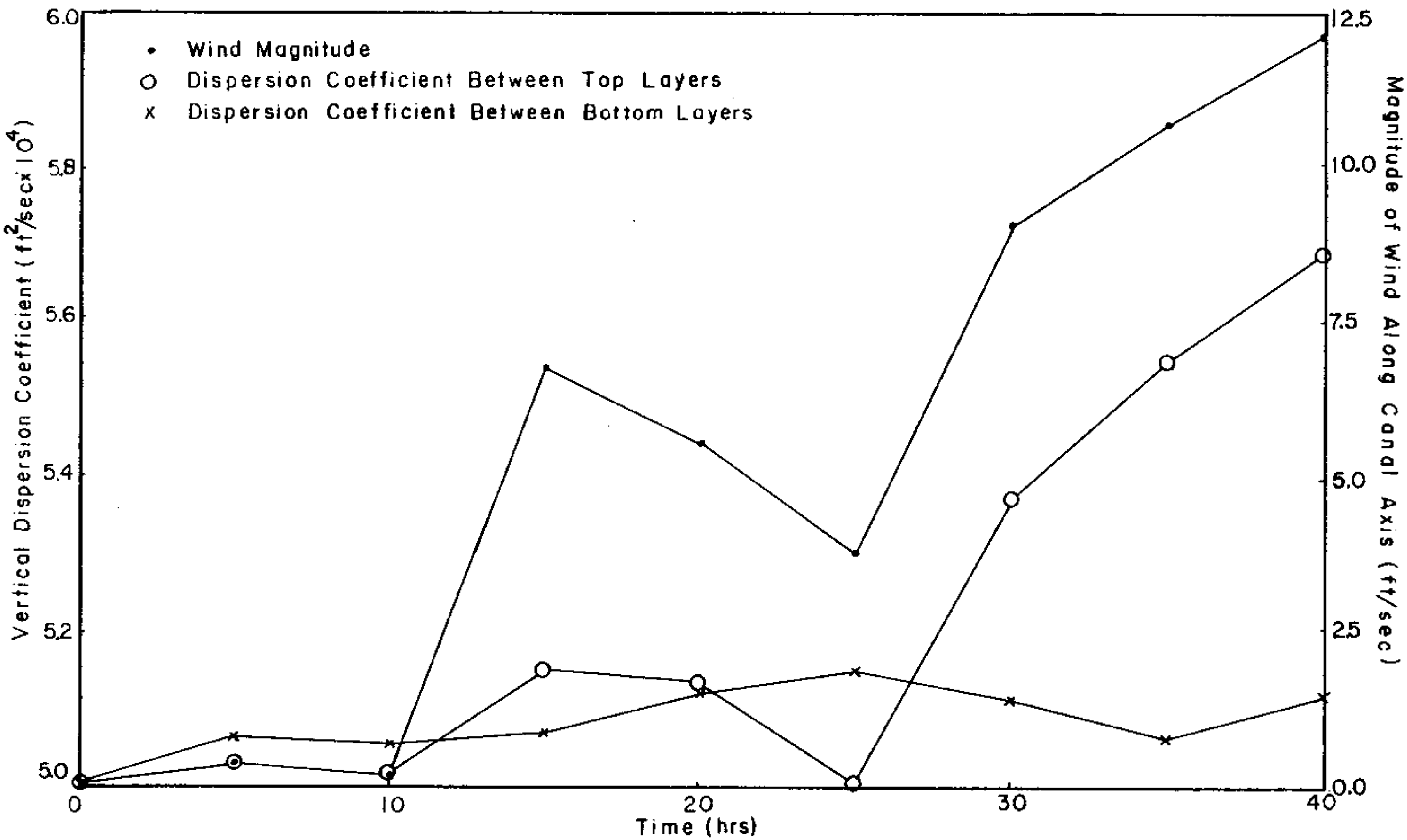


Figure 8.6 - Variation of Vertical Dispersion Coefficient With Time and Wind Speed at Mid-Point of Reach 1 (Figure 8.1), 57 Acres October, 1977 Case History.

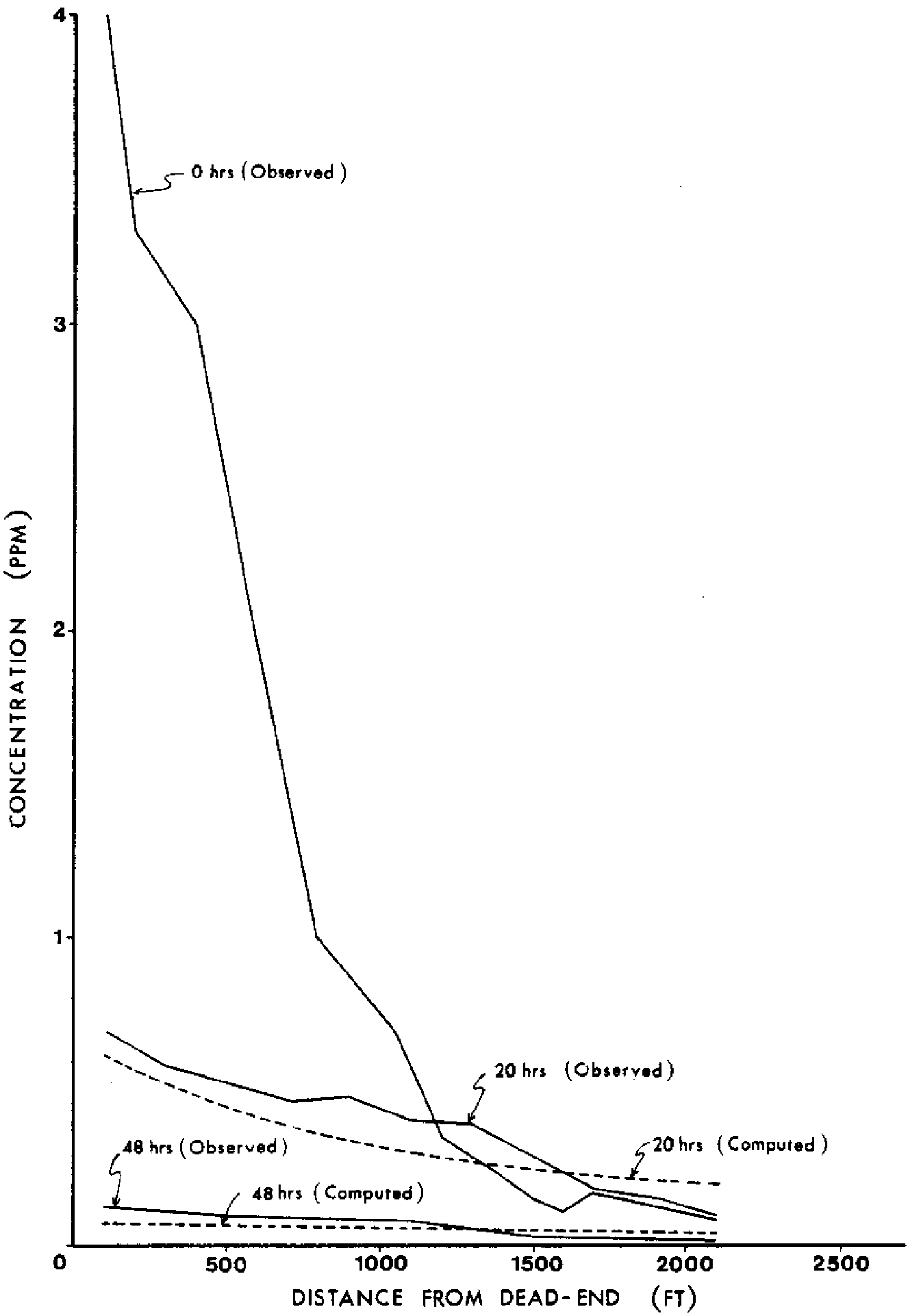


Figure 8.7 - Comparison Between Observed and Computed Concentration for Loxahatchee River North Canal, June, 1977.

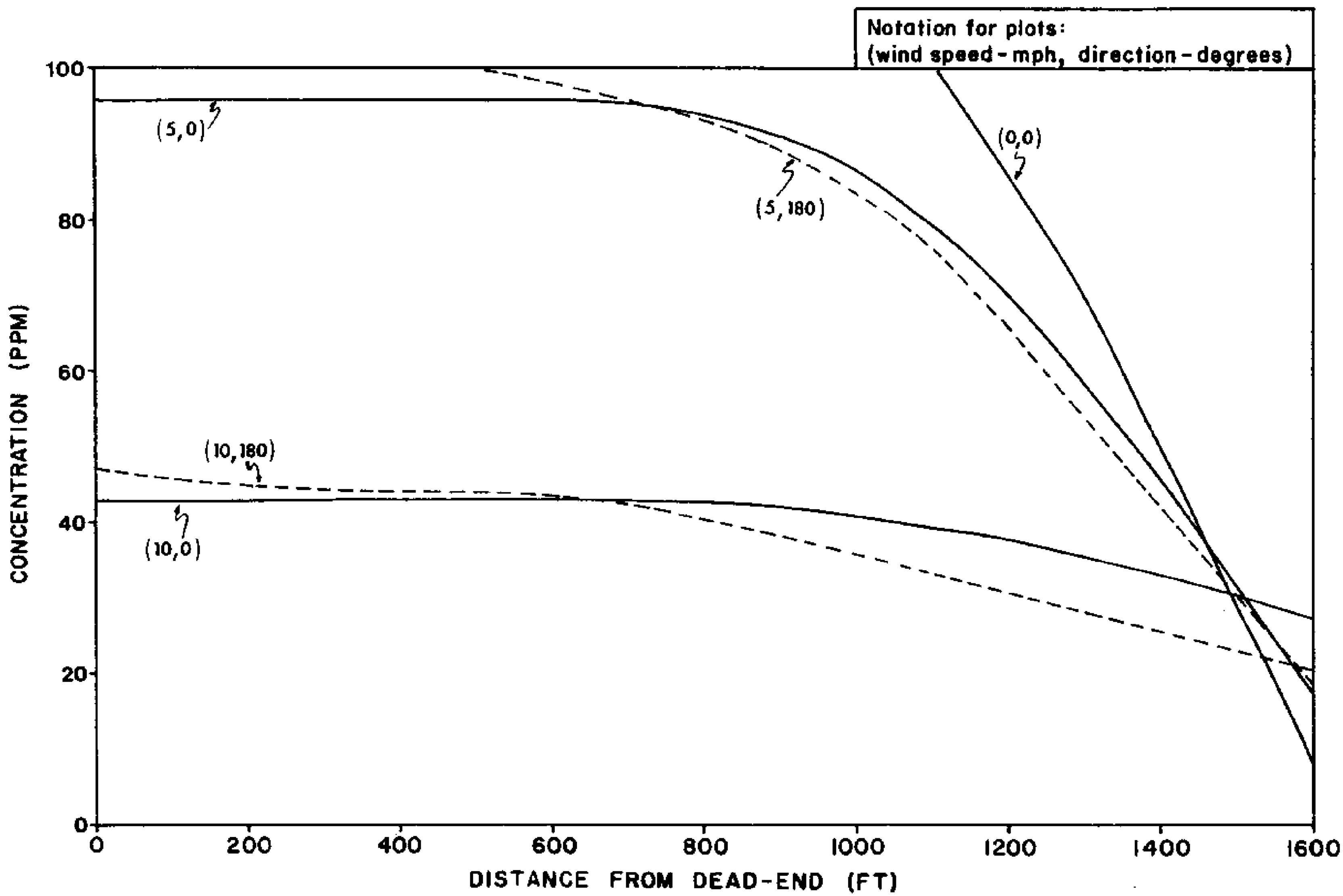


Figure 8.8 - Case 1W: Initial Concentrations, c_i = 100 ppm, Background Concentration, c_{RW} = 5 ppm - Ten Tidal Cycles.

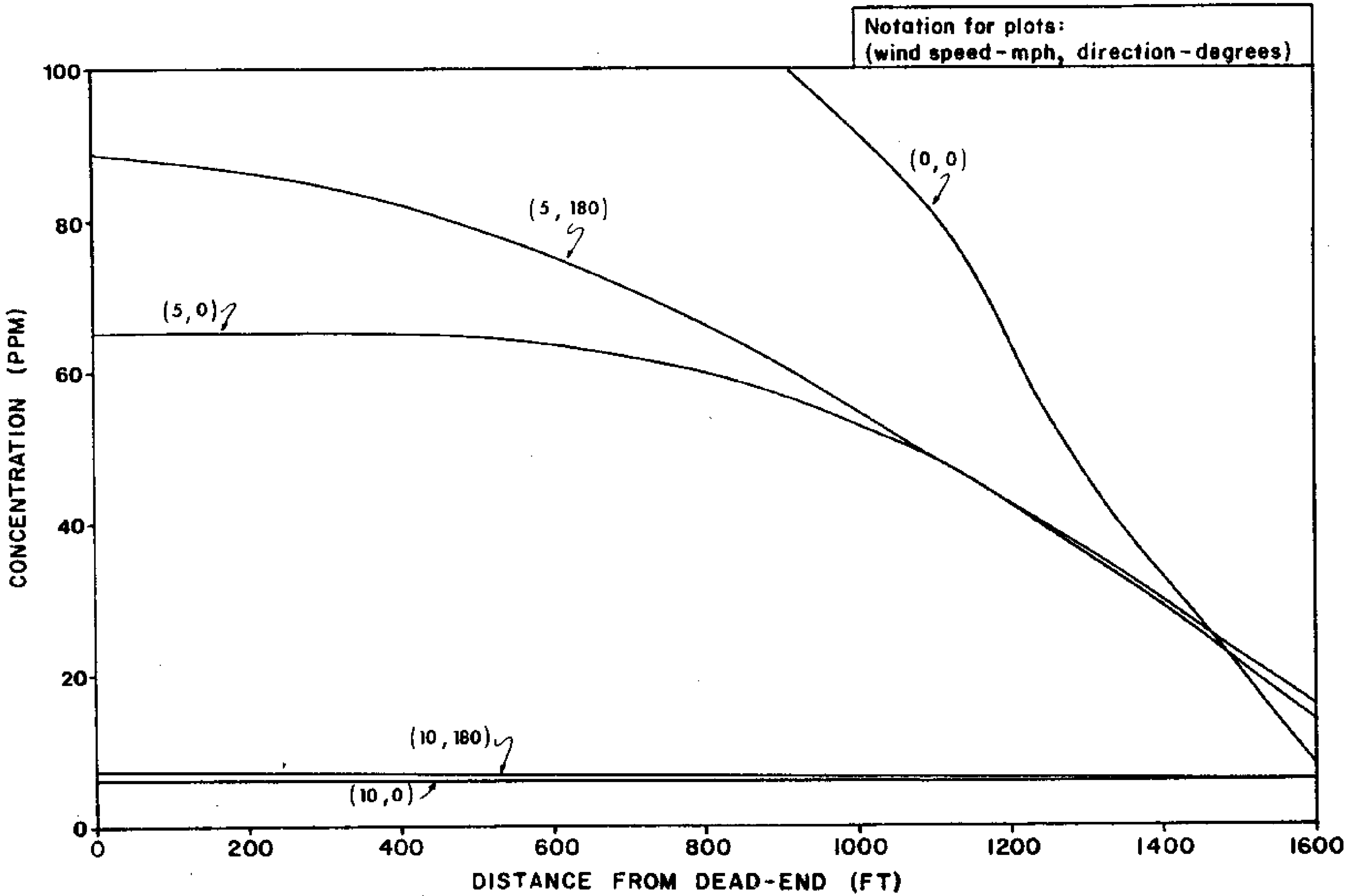


Figure 8.9 - Case 1W: Initial Concentrations, c_i = 100 ppm, Background Concentration, c_{RW} = 5 ppm - Fifty Tidal Cycles.

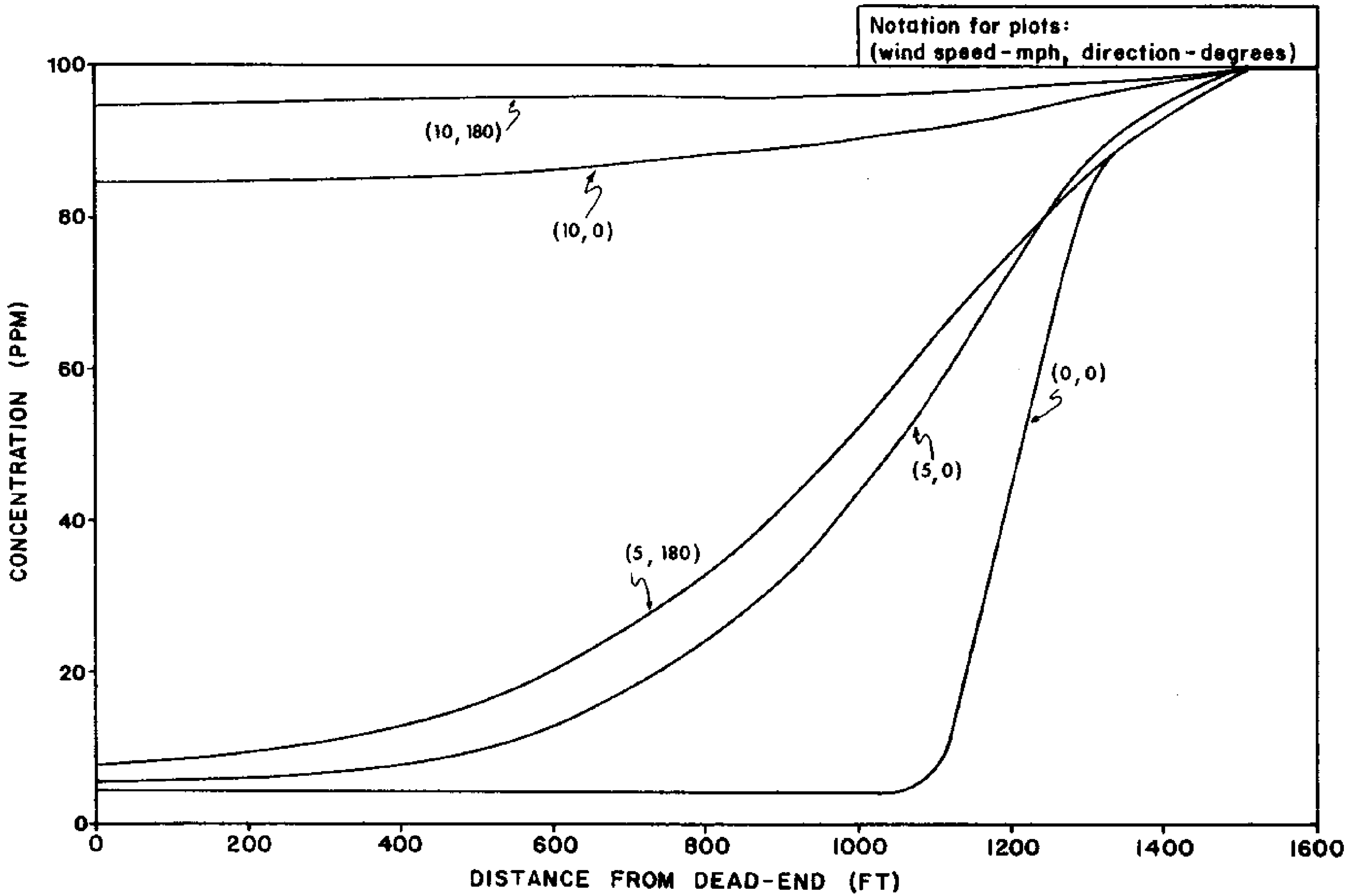


Figure 8.10 - Case 2W: Initial Concentrations, c_i = 5 ppm, Background Concentrations, c_{RW} = 100 ppm - Ten Tidal Cycles.

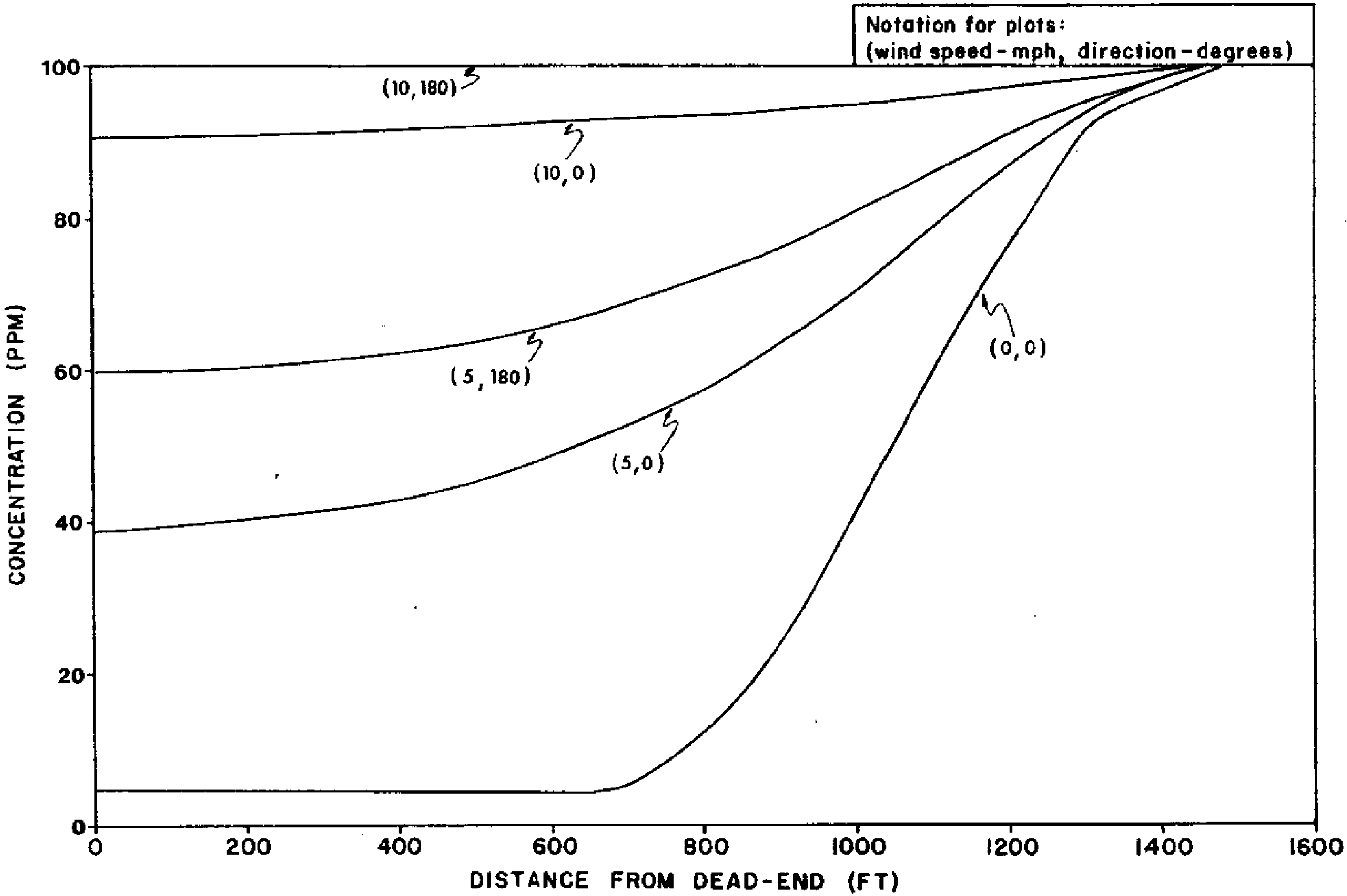


Figure 8.11 - Case 2W: Initial Concentrations, c_i = 5 ppm, Background Concentration, c_{RW} = 100 ppm - Fifty Tidal Cycles.

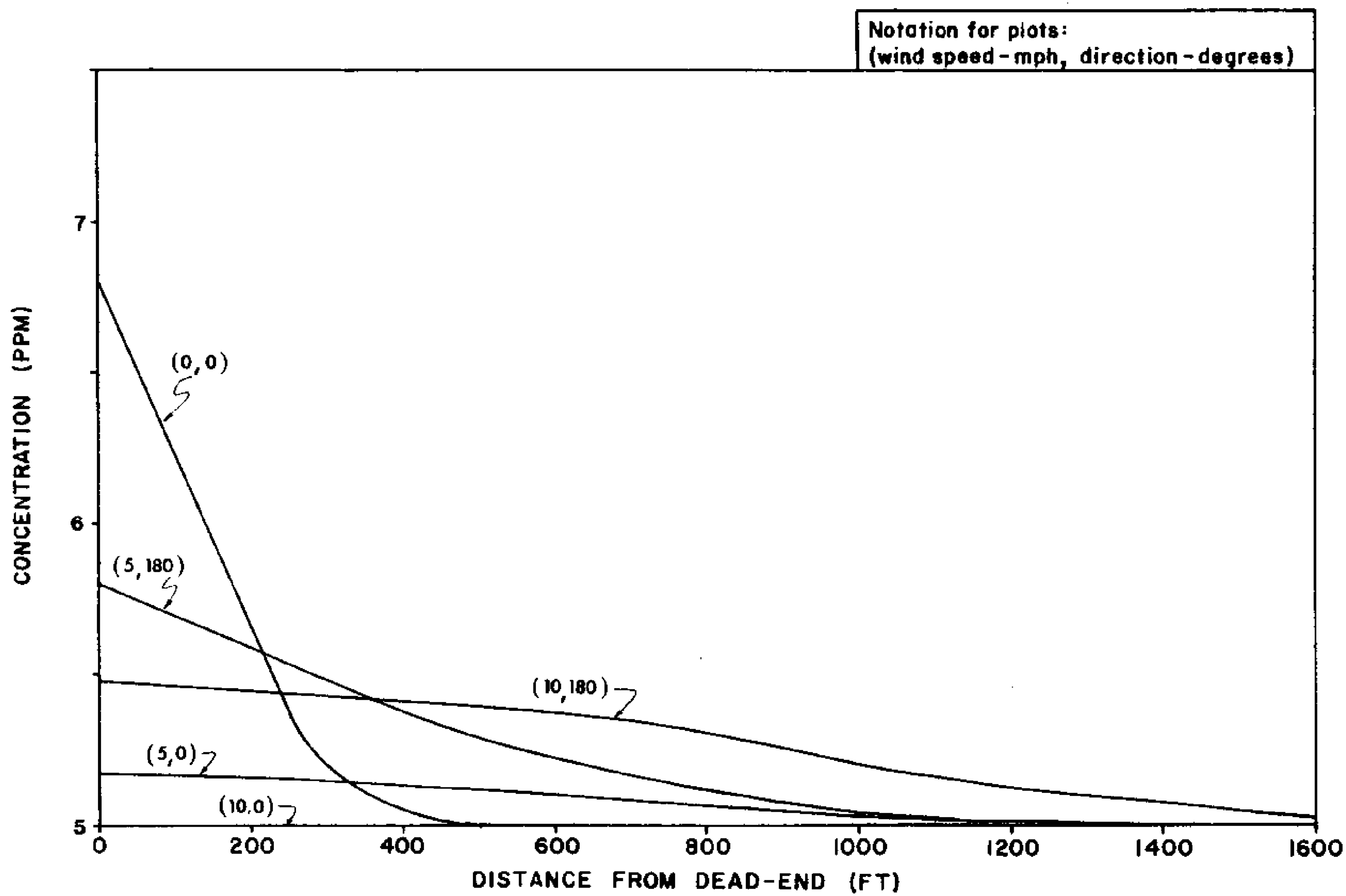


Figure 8.12 - Case 3W: Lateral Inflow Distribution Along Length of Canal - Ten Tidal Cycles.

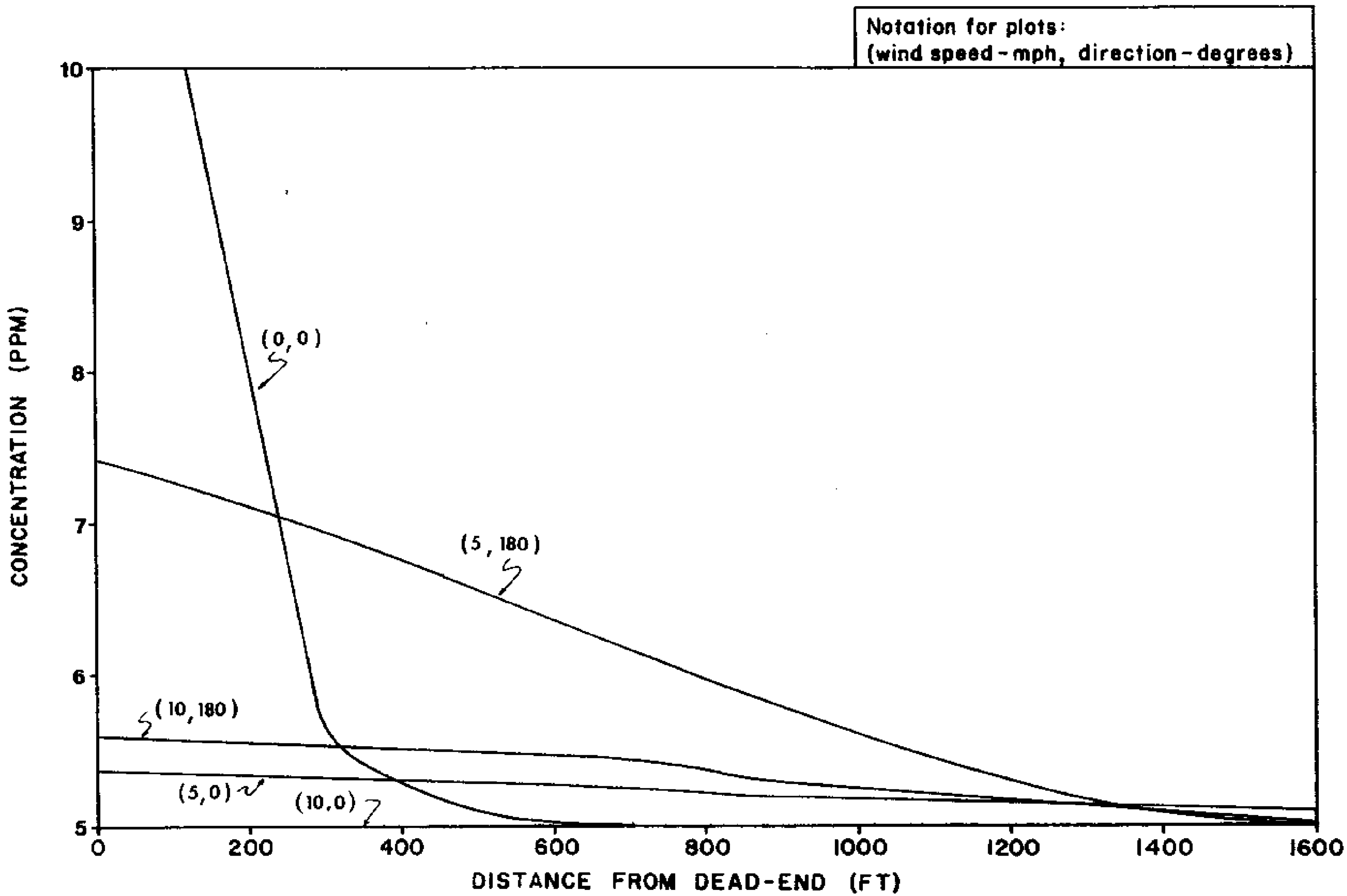


Figure 8.13 - Case 3W: Lateral Inflow Distribution Along Length of Canal - Fifty Tidal Cycles.

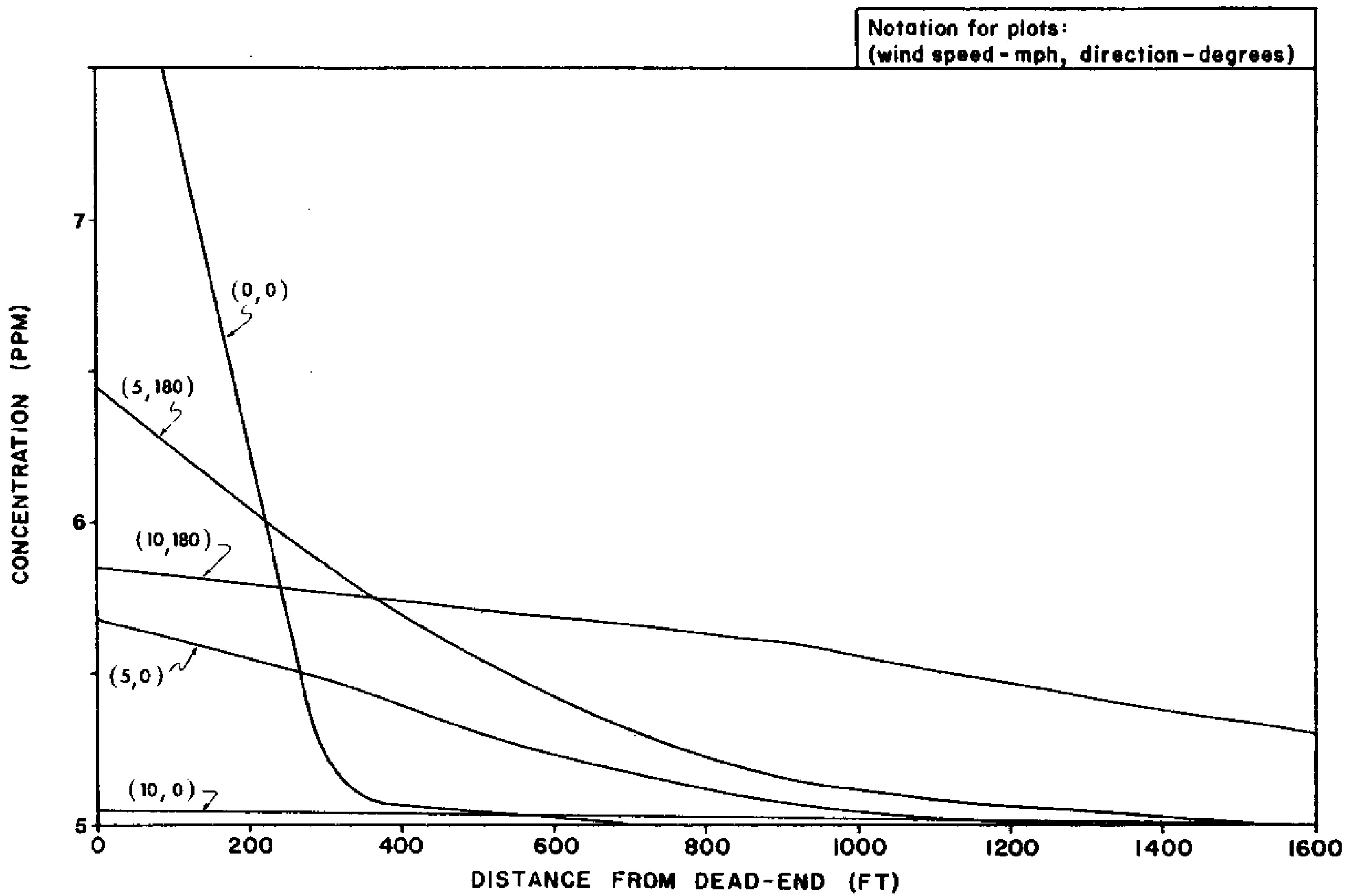


Figure 8.14 - Case 4W: Lateral Inflow Distribution Along Upper 1/2 of Canal - Ten Tidal Cycles.

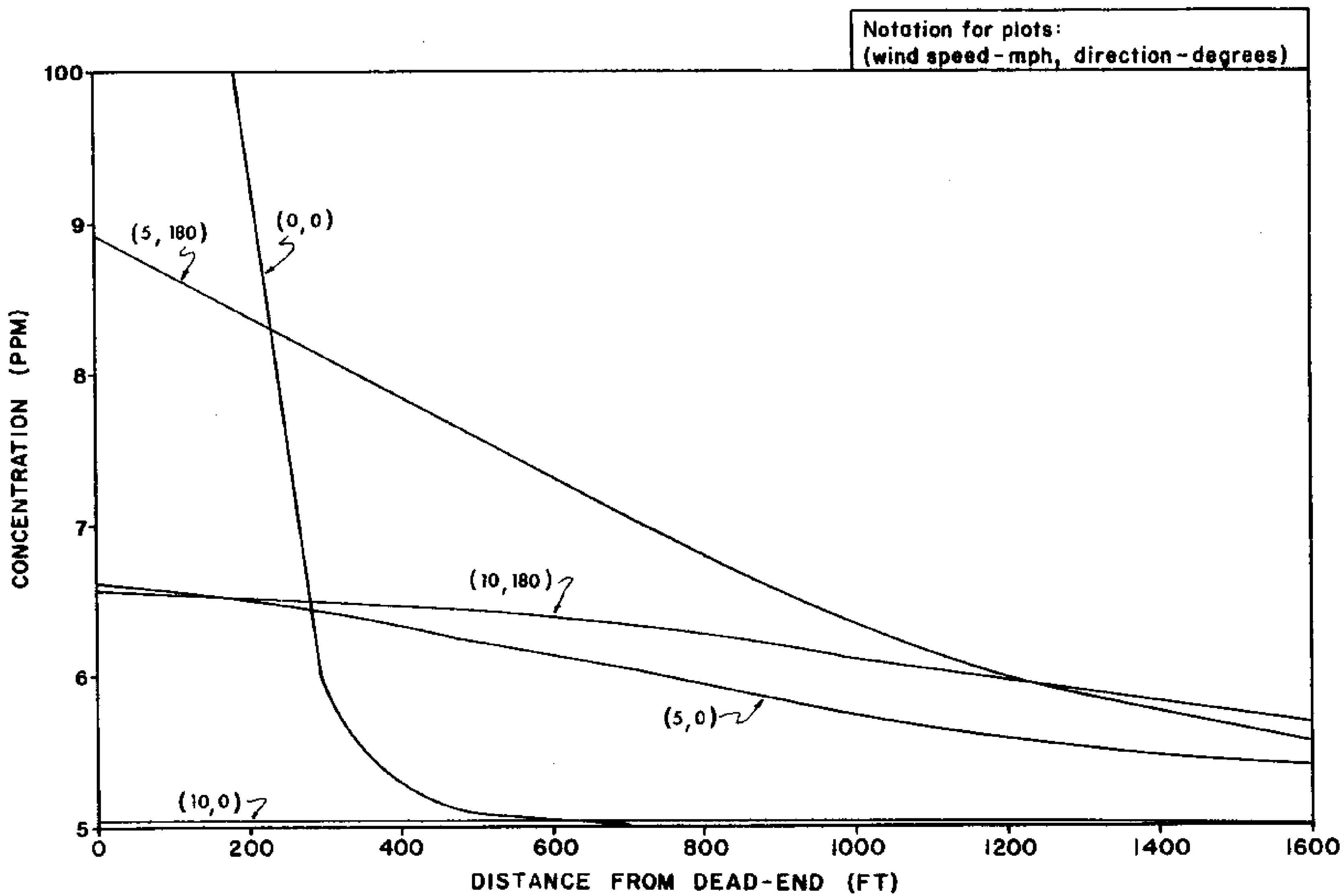


Figure 8.15 - Case 4W: Lateral Inflow Distribution Along Upper 1/2 of Canal - Fifty Tidal Cycles.

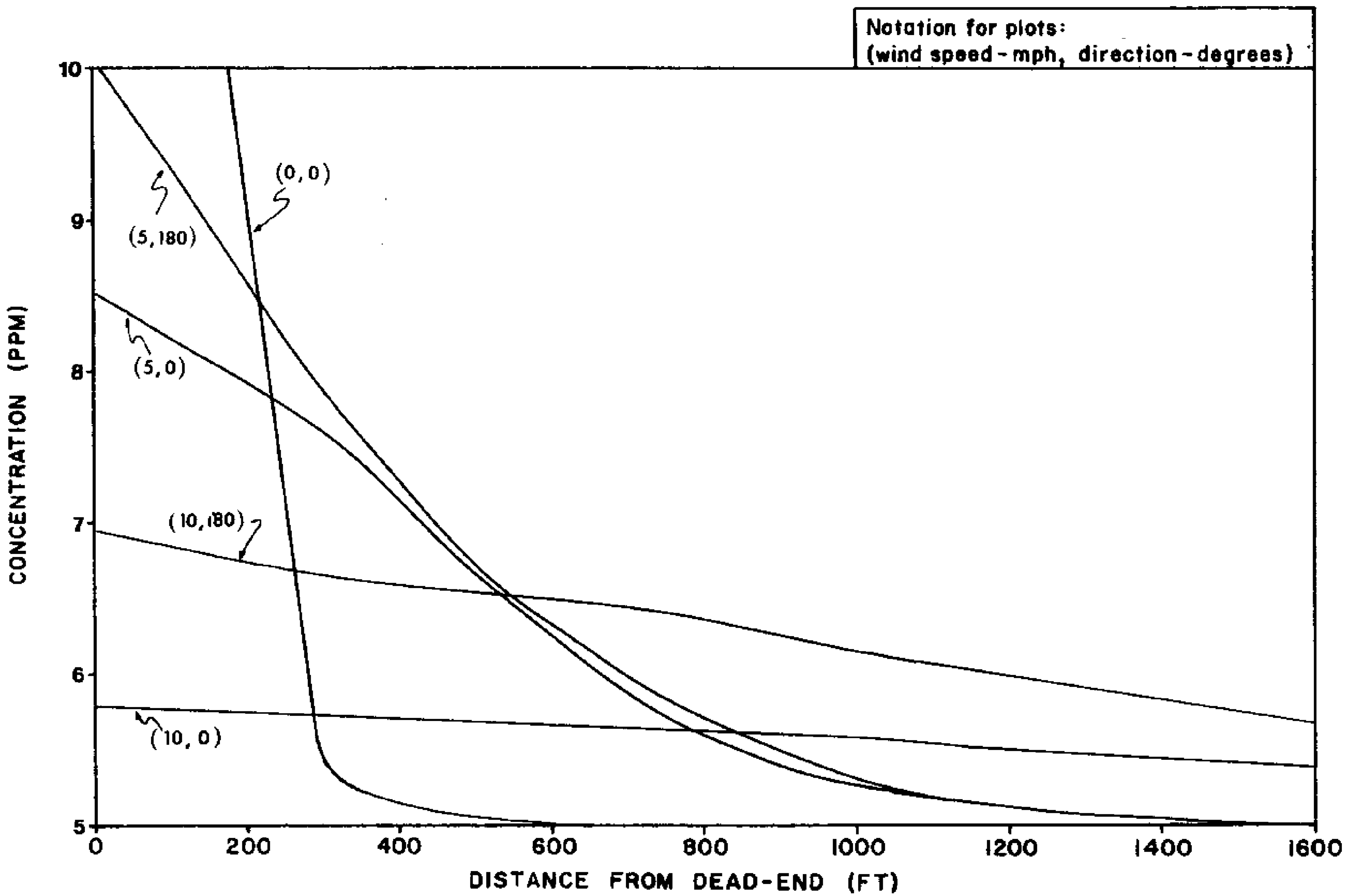


Figure 8.16 - Case 5W: Lateral Inflow at Dead-End - Ten Tidal Cycles.

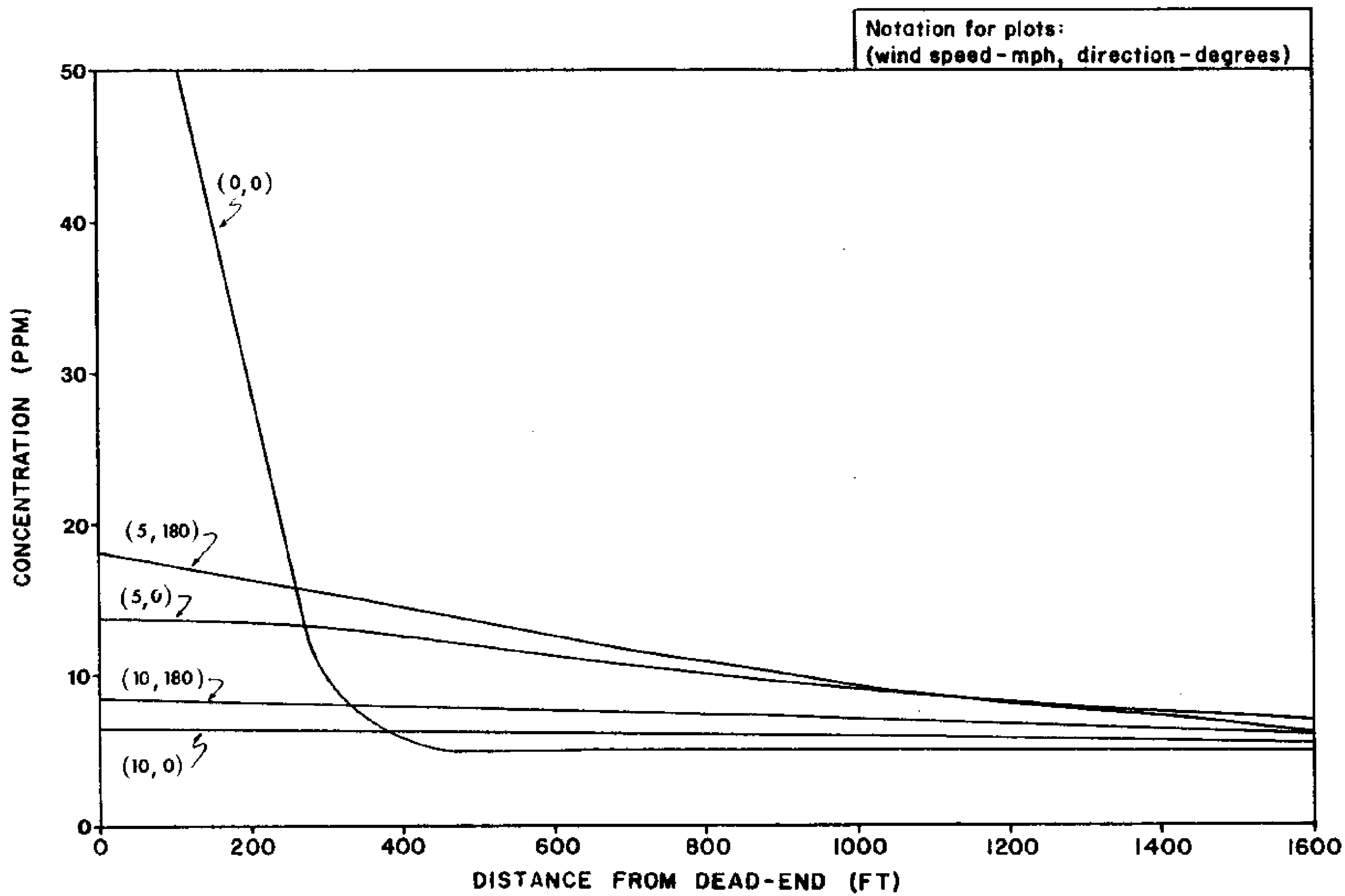


Figure 8.17 - Case 5W: Lateral Inflow at Dead-end - Fifty Tidal Cycles.

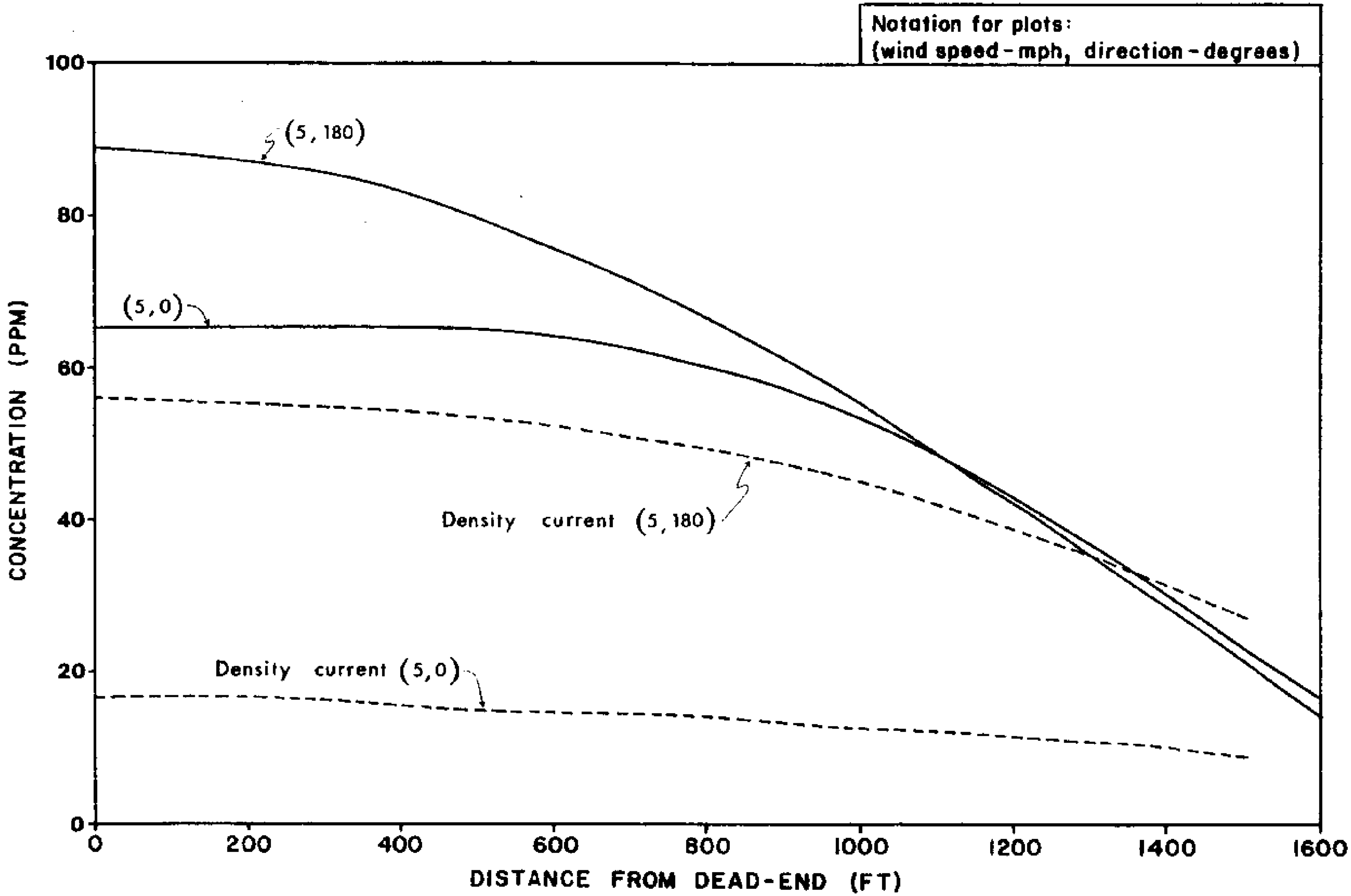


Figure 8.18 - Case 1S: Effect of Salt Wedge With Initial Concentration, c_i = 100 ppm, Background Concentration, c_{RW} = 5 ppm - Fifty Tidal Cycles.

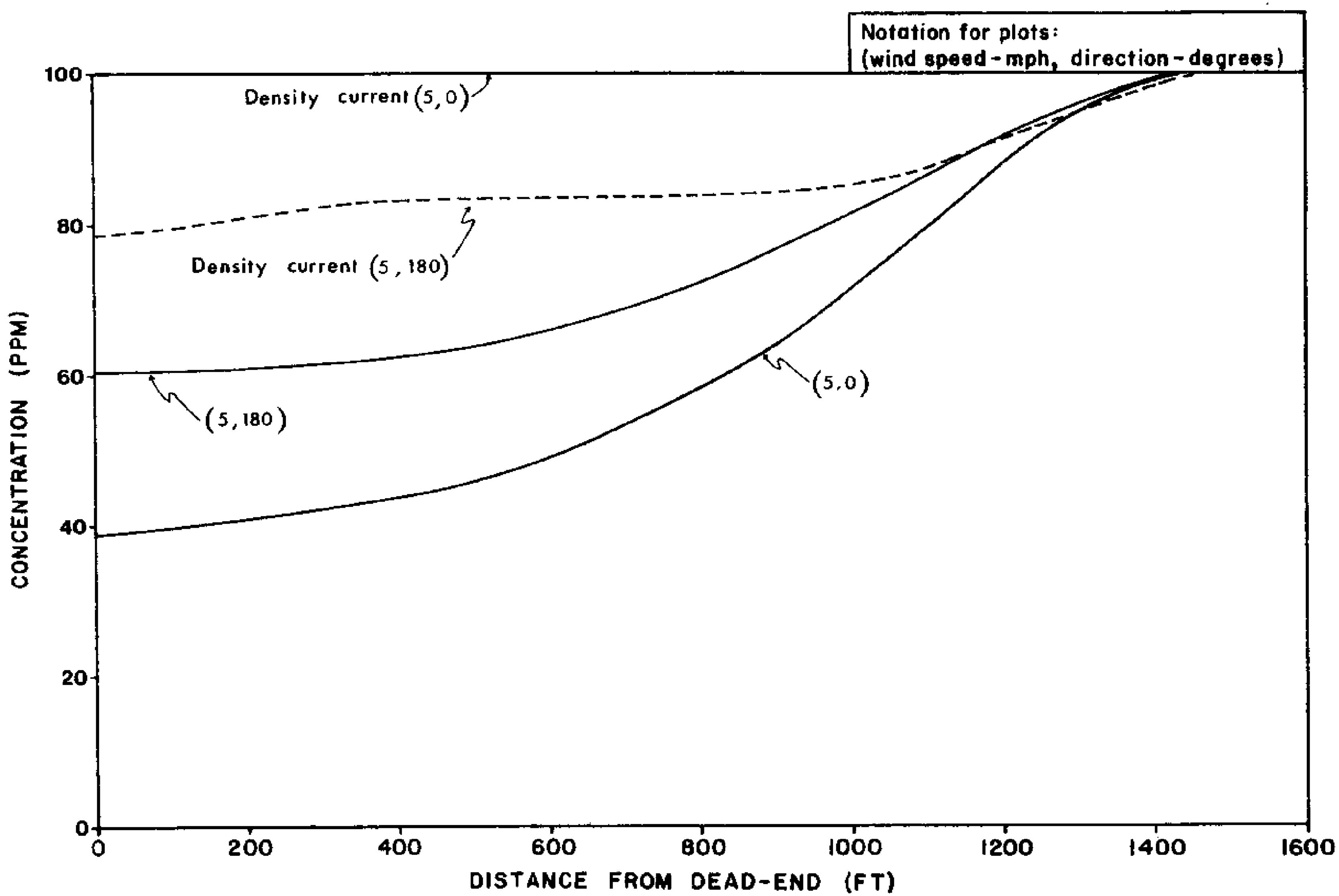


Figure 8.19 - Case 2S: Effects of Salt Wedge With Initial Concentrations, c_i = 5 ppm, Background Concentration, c_{RW} = 100 ppm - Fifty Tidal Cycles.

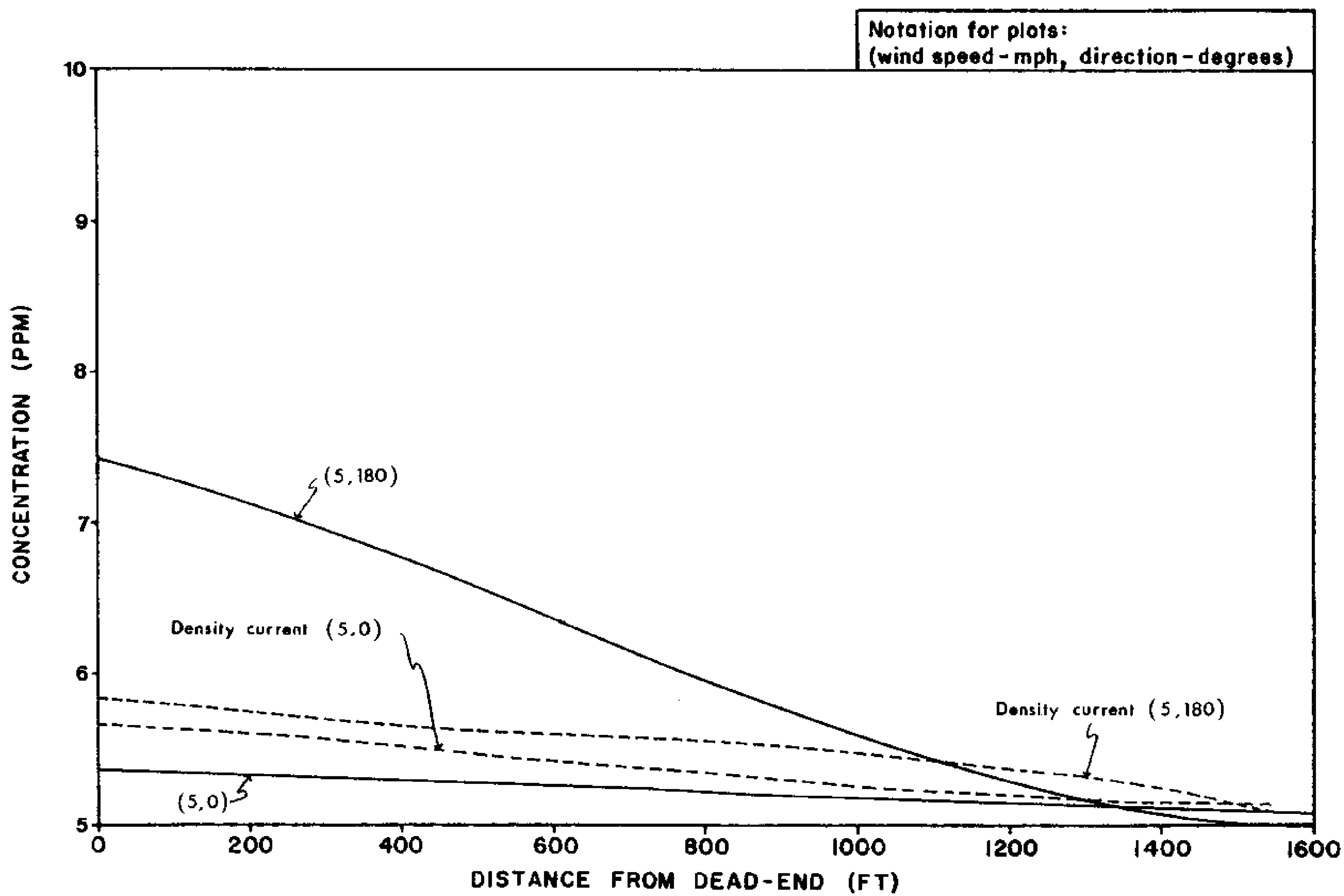


Figure 8.20 - Case 3S: Effect of Salt Wedge With Lateral Inflow Distribution Along Length of Canal - Fifty Tidal Cycles.

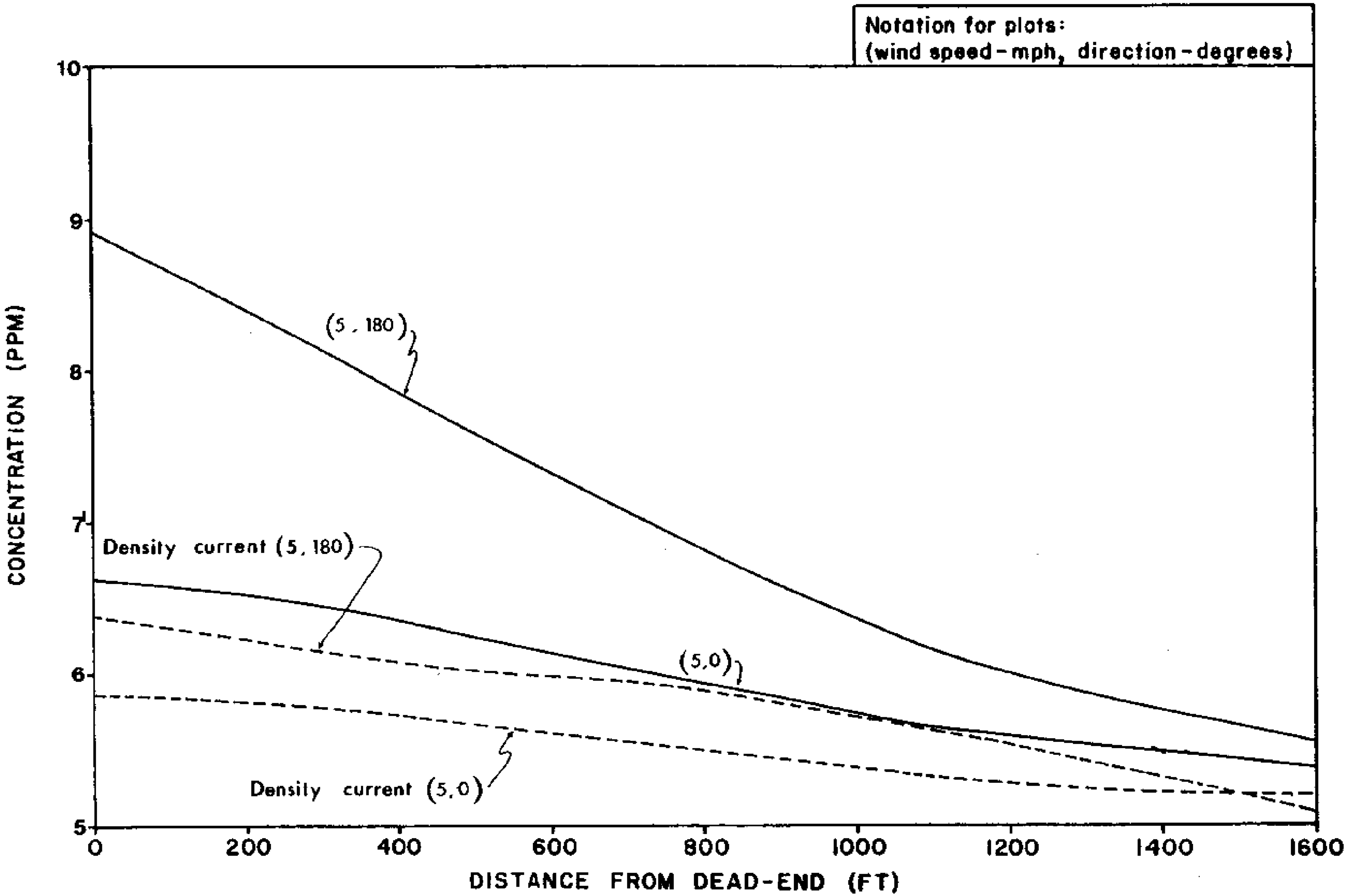


Figure 8.21 - Case 4S: Effect of Salt Wedge With Lateral Inflow Distribution Along Upper 1/2 of Canal - Fifty Tidal Cycles.

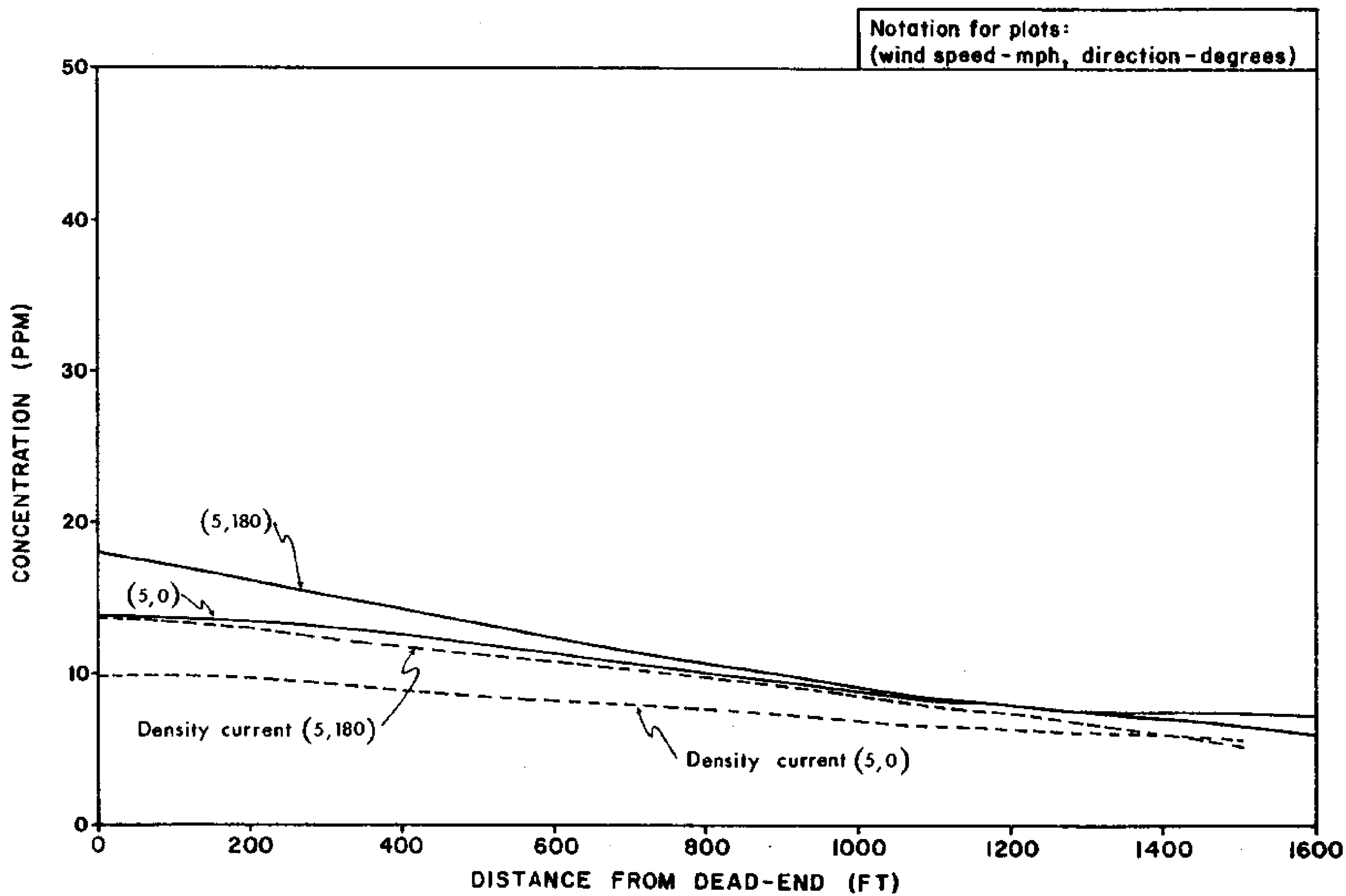


Figure 8.22 - Case 5S: Effect of Salt Wedge With Lateral Inflow Distribution at Dead-End - Fifty Tidal Cycles.

Table 8.1 - Parameters for Big Pine Key Canal III Case History

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 right bank, s_R	0 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 lateral, K_y vertical, K_z	 0.1 0.001 0.008
Background dispersion coefficient, E_o	0 ft^2/sec
Vertical momentum transfer coefficient, N_z	0.005 ft^2/sec
Background concentration, c_{RW}	0 ppm
Constant wind speed, w_s	5 mph
Constant wind direction	0 degrees

Table 8.2 - Parameters for 57 Acres Case History

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 8.2 - 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 8.3 - Parameters for Loxahatchee North Canal Case History

PARAMETER	VALUE
Length, L	2200 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	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 (4.64), u_4	3000
Density of freshwater, RHOF	1.94 slugs/cu ft
Density of saltwater, RHOS	1.99 slugs/cu ft

Table 8.4 - 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	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
Depth to saltwater interface**, DSM	5 ft
Coefficient of Equation (4.64)**, 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 8.5.2).

Table 8.5 - Variability Studies Using Three-Dimensional Model

CASE NUMBER	VARIATION
1W*, 1S	Initial concentration, c_i = 100 ppm Background concentration, c_{RW} = 5 ppm
2W, 2S	Initial concentration, c_i = 5 ppm Background concentration, c_{RW} = 100 ppm
3W, 3S	Lateral inflow rate, q_I = 0.04 cu ft/hr/ft along whole length of canal. Inflow concentration, c_I = 100 ppm
4W, 4S	Lateral inflow rate, q_I = 0.08 cu ft/hr/ft along upper 1/2 length of canal. Inflow concentration, c_I = 100 ppm
5W, 5S	Lateral inflow rate, q_I = 0.32 cu ft/hr/ft in 200 ft adjacent to dead-end. Inflow concentration, c_I = 100 ppm

* W - refers to wind induced circulation only (Section 8.5.1).
S - refers to density induced circulation (Section 8.5.2).

CHAPTER 9

SUMMARY OF NUMERICAL MODELING

9.1 Summary of One-Dimensional Modeling

As an initial step in the development of a numerical model to accurately, and inexpensively simulate mass-transport in low energy tidal finger canal networks, a simple one-dimensional numerical model was developed based on the horizontal water surface assumption and the one-dimensional, mass-transport equation. The horizontal water surface assumption was significant in that it greatly reduced the complexity of the governing equations, thus making solution techniques simpler and cheaper. As shown in Section 3.1.3, this assumption is reasonable when compared with field measurements and another numerical model based on the full equations of continuity and momentum (or the dynamic equation). Being able to model the velocity field using the continuity equation alone, it was found that velocities in the canal network could be obtained as closed form functions of the longitudinal displacement, x, from the dead-end of each reach.

Furthermore, as shown in Section 3.1.4, using a classical form of the longitudinal dispersion coefficient, E_ℓ, this coefficient could also be found as a closed form function of x. This greatly simplified the problem, and by including a lateral inflow term $q_I c_I/A$ for the rate of production or loss term of Equation (3.15), the number of unknowns was reduced to one, the concentration, c.

However, upon programming this model using the more common finite-element and finite-difference techniques, such as the central-difference explicit method, it was found that a severe damping term was present in the schemes. This term is called numerical dispersion, and like natural dispersion is a second order effect produced when infinite Taylor series expansions to the first order time derivative and the first order spatial derivative are truncated after the second term.

Analysis of this problem, and an extensive literature review, showed that for economic choices of the time increment, Δt, and the spatial increment, Δx, the numerical dispersion term can have a significant effect in systems in which the natural dispersion coefficient is small. In Floridian canals, and other low energy canal networks around the Gulf Coast and along the Eastern Atlantic Seaboard, measured maximum dispersion coefficients are on the order of 5 ft^2/sec. This is very small when compared with dispersion coefficients in rivers and estuaries, which are often two orders of magnitude larger. In these canals, the numerical dispersion inherent in most common finite-element and finite-difference techniques can swamp the natural dispersion being modeled giving the damped concentration profiles found in early models.

The main problem, then, in a one-dimensional analysis of mass-transport in tidal networks, and indeed one of the major research efforts of the entire project, became the developing of a numerical technique which would eliminate, or at best greatly reduce the amount of numerical dispersion in the method, so that error terms would be small compared with the longitudinal dispersion term. A variety of methods were investigated as discussed at length in Chapter 3.

Two method-of-characteristics techniques were developed, one using a fixed grid structure and the second using a movable grid system. The latter method was dropped because it was found that when lateral inflow was being modeled, the grid system tended to drift out of the canal with the net positive velocity induced by the inflow. The fixed grid method-of-characteristic technique appeared to be much more promising at first, until tests showed that the interpolation routine between grid points could not handle curvature in the concentration profile without damping the results.

Another technique extensively investigated was an analog model approach using the hydrid facilities of Martin-Marietta Aerospace in Orlando, Florida. Investigation of the theoretical form of the numerical dispersion term for uniform flow conditions showed that the numerical dispersion might be reduced to zero as the time step, Δt, was made smaller. An analog machine models time as a continuous variable, and encouraging results were obtained from a model using finite-difference forms of the spatial variables. Unfortunately, the conclusion of the research showed that the method could not be realistically extended to a two- or three-dimensional model because of severe hardware limitations, and that the nodal structure of the model was fairly inflexible.

The continuing research effort finally produced two numerical models which satisfied the objectives being sought. The first model used a second upwind differencing technique with limited anti-dispersion and flux corrected transport [Lee, 1977]. As was seen in

Section 3.5, the method was very successful in minimizing numerical dispersion and gave accurate results fairly inexpensively. The second model used a variation in the method of second moments, to incorporate the change in the depth of flow with the tide. The results obtained were equally as accurate as produced by the first model, and the runs required slightly less CPU time.

Comparing the two techniques, it was decided to use the method of second moments to develop a three-dimensional numerical model. The reasons for choosing this technique were that it seemed more physically meaningful to model conservation of moments than to continually make corrections for errors, and that its formulation was simpler, particularly in junctions.

Once a one-dimensional model had been developed, it was used to analyse the effects of changing geometric parameters of the canal layout (see Section 3.8). Most of the results presented in that section are intuitively obvious, and usually showed that equilibrium concentration profiles were lower when a bigger volume of canal water was present to dilute incoming lateral inflow. This analysis also clearly showed that a one-dimensional model was not sufficient to adequately simulate prototype conditions. For small tidal amplitudes, the exchange volume with the receiving waterbody is small and very little flushing is obtained in the canal. The other interesting result was that the tidal entrance decay coefficient, τ, could be varied through a fairly wide range of values without significantly affecting the equilibrium concentration profile within the canal. This result was gratifying because the tidal entrance boundary condition is not well understood and many modelers assume a decay type condition during flood tide. The insensitivity of the parameter means that its value can be assigned from field measurements without the need for an exhaustive analysis in that region of the canal network.

The one-dimensional model was adequate for showing the qualitative effect of varying geometric and inflow parameters, results which would be expected to be duplicated using a two- or three-dimensional model. However, as the variability study of the tidal amplitude showed, a one-dimensional mass-transport model is not adequate in simulating prototype conditions influenced by wind and density currents.

This point was illustrated in Section 8.2, in which an EPA study of the Big Pine Key Canal III was analysed [EPA, May, 1977]. In simulating the observed transport of a dye cloud introduced into the canal using the CRM, the dispersion coefficient in the model was reduced by a factor of 5. In fact, the SWMM model was first used, but was found to suffer from numerical dispersion, and could not predict the passage of the center of mass of the dye cloud.

In testing his model, Lee [1977] also tried to model this canal and found that he had to increase his dispersion coefficient by two orders of magnitude. This result clearly indicated that SWMM and CRM are not suited to investigations of transport characteristics in low energy tidal canals, and also that a one-dimensional investigation of these systems is meaningless. Lee's model could not predict the movement of the center of mass of the dye cloud either, as theoretical tidal induced velocities were on the order of 0.001 ft/sec.

The conclusions arrived at from this initial study were that at least a two- and possibly three-dimensional model would be required to model the transport characteristics of these canal systems.

The tidal flux in the canal system was seen to be often one of the least important phenomenon in short canals, and that the important factors such as wind and density induced circulation needed to be incorporated into such a model. The second conclusion was that the method of second moments was well suited for an extension to such a model and was chosen over the other methods investigated.

9.2 Summary of Three-Dimensional Modeling

In Chapters 4, 5 and 6, a three-dimensional mass-transport model was developed to simulate the transport characteristics of low energy tidal canal networks. The model incorporated not only the tide, but also wind induced circulation, density currents, lateral inflows and decay. The numerical technique used was an extension of the method of second methods outlined in Section 3.6.

The extension to the three-dimensional model, was made by giving the cell structure of the model the capability of expanding and contracting with the tidal flux. By allowing the cross-sectional areas of each layer to "flex" in proportion to the volume flux through the layer, and then allowing the lateral cells in each vertical layer to change in cross-sectional areas depending on the volume flux through them, a model was developed in which the cell sturcture could change its shape in response to the change in tidal elevations and velocities, and in which mass-transfer could be modeled by only considering the exchange between longitudinally adjacent cells in the same vertical layer. The model uses a longitudinal dispersion velocity, added to a convective transport term, and lateral and vertical dispersive transfer using a similar approach. Decay was introduced on an individual cell basis, or as a uniform coefficient for the whole reach.

The various component parts of the velocity field were identified as unique closed form functions, and their forms superimposed to synthesize the model velocity field. The tidal component, as for the one-dimensional model, was derived using the horizontal water surface assumption, but was assumed to have a vertical logarithmic distribution. The magnitude of the tidal flow was obtained using an upstream volumetric tidal prism incorporating the lateral inflow rate. Secondary currents in bends were derived from Rozovskii's theory [1957] of logarithmic flow over a rough bed. A wind induced vertical velocity profile was theorized by simplifying the momentum equation, using a parabolic distribution with a flow reversal at the one-third depth from the water surface. Finally, a density current, induced by the movement of a saline wedge in a freshwater canal, was introduced by making an analogy with the passage of a flood wave going over bank in a river. The movement of a triangular wedge is governed by the dynamic response to the rate of rise of the saltwater interface in the receiving waterbody during the flood tide, and by the kinematic wave produced as the salt wedge in the canal loses potential energy during the ebb tide.

In the model, junctions were treated as transition zones between flow conditions in adjoining reaches. Only conservation of mass, is modeled in junctions, and dispersion is ignored because of the inherent numerical dispersion present in this region.

Chapter 7 outlines stability and convergence criteria for the model. On the whole, the model is stable and has good convergence characteristics. The problem areas are the interfacing of schemes at the junctions, and in the modeling of density currents. Although instabilities were generally not a problem, particularly for nondensity flow simulations, the case of uniform lateral inflow along the length of a test canal influenced by the movement of a saline wedge resulted in instabilities which were of a sufficient magnitude to affect the very small concentration differences being modeled. However, in all other cases, the model performed well.

The model was used to first simulate the movement of a dye cloud in the Big Pine Key Canal III [EPA, May, 1977], and the results clearly showed the power of a two-dimensional formulation of the velocity field. Using values for the model parameters which were consistent with other case studies, the observed longitudinal concentration profiles through time were accurately simulated using an estimated value for wind speed. With the wind induced circulation being incorporated into the model, not only the peak concentration values could be accurately reproduced but also the center of mass of the distribution.

Other case histories simulated observed conditions in the 57 Acres canal network and the Loxahatchee River site, both located on the East Coast of Florida. The first site was considered to be influenced primarily by wind induced circulation, whereas salinity measurements at the second site showed the existence of a saltwater interface. In modeling these systems, good agreement was found as discussed in Chapter 8. More importantly the agreement was achieved without altering the values of the vertical momentum transfer coefficient, N_z, the dimensionless vertical dispersion coefficient, K_z, and the background dispersion coefficient, E_0, between the sites.

In all of these case histories, values of N_z = 0.002 ft^2/sec, K_z = 0.0001 and E_0 = 0.0005 ft^2/sec were used. For the Loxahatchee River Site, in which density circulation was included, a value of 3000 was determined for the coefficient, u_4, of Equation (4.64).

The fact that the various systems could be modeled with uniform values of the major parameters was the important finding of the study. It shows that a more physically correct approximation to prototype transport phenomena is being modeled, a process which cannot be incorporated into a one-dimensional model. However, the need to use a background dispersion coefficient, E_0, shows that there is still much to be learned about the diffusion process.

The final part of Chapter 8 was a variability analysis of the effects of wind induced circulation and density currents. The results showed that a wind blowing down a canal (in the positive x-direction) was more efficient in flushing a canal and resisting the entry of polluted receiving waters than a wind blowing up the canal from the tidal entrance towards the dead-end. Furthermore, the addition of a density current to the system increased the rate at which the canal flushed, but also increased the rate at which canal water was exchanged with the receiving waterbody.

The variability analysis showed only a qualitative effect. It would be dangerous to base a proposed scheme on the basis of the results obtained from a variability analysis without first undergoing an extensive analysis based on the actual conditions present at the existing or proposed site. Once wind, salinity and tidal elevation data have been obtained, the canal layout can be varied in a systematic manner to best utilize the available energy sources.

9.3 Future Research

The three-dimensional, mass-transport model presented in this report was developed to simulate the transport properties of low energy tidal canal networks. The aim of the study was to show that a model of this nature was required to accurately model field conditions. This has been clearly demonstrated in this report, and thus future research should be centered around expanding this model to improve its versatility and scope of application. Also, improvements in the numerical techniques used in the model could be studied.

The aspects of the model which might bare further examination are the matching of numerical schemes, the method of second moments and the upwind difference method, at junctions and in modeling density currents. The junction is a very complex area of the canal system in which transitions are occurring between the various adjoining reaches. It may prove to be very difficult to accurately model this area without resorting to a fully three-dimensional model. However, the density current theory might be improved by considering the change in centers of mass and widths of distributions in cells in which an upwardly induced flow is present to conserve mass. Also, the condition through junctions and at canal dead-ends might be further investigated, as instabilities appear to originate in these areas.

The present model was only able to model flow in loops, or with multiple tidal entrances, in situations where the flows came together again at a null point. The model cannot handle the case in which the flows recombine and enter an upstream reach. This is because at a junction, one downstream reach must be specified. In the case described above, two reaches can be considered to be downstream reaches for that junction, but one of them has to be specified as a left or right branch. With the present layout of the model, in which the cells are numbered from upstream to downstream in each reach, the lower numbered cells of a reach adjoin an upstream junction, but the higher numbered cells of a branch canal adjoin the same junction. Thus, in effect, the "second" downstream reach is defined the wrong way around, and some additional work to rectify this problem is required.

Further desirable extensions of the model include the more obvious ones of being able to model not just one isolated substance, but combinations of substances including chemical and biochemical interreactions. The incorporation of a water quality package would give the model the ability to simulate the major transport and quality phenomena present in canal networks. However, the danger of such a compound model, as mentioned in the introductory chapter, is that the number of unknown parameters is dramatically increased, and unless a careful field calibration study is undertaken by experienced scientists, there is the possibility of producing numbers which appear to be correct but which in reality have no relationship to actual conditions. The lesson to be learned from the comparative analysis with one-dimensional, mass-transport models, is to model only those phenomena whose effects are well understood and which play an important part in the overall process, and to avoid confusing the results with a multitude of factors whose sum effect could give entirely erroneous results.

9.4 The Numerical Model as a Design Tool

CANNET3D has been designed to be user oriented and incorporates few parameters which need to be calibrated. In fact for most practical applications, the longitudinal and lateral dispersive terms play an insignificant part in changing concentration values. The major parameters are the vertical momentum transfer coefficient, N_z, the dimensionless vertical dispersion coefficient, K_z, the background dispersion coefficient, E_0, the dimensionless density current coefficient, u_4 (defined in Equation (4.64)), and the tidal entrance decay coefficient, τ.

Although in the case histories examined in this report, values for these parameters varied little between the different sites, it is recommended that a field study be conducted to determine the actual values that would apply to a unique location. The values of these parameters together with measured wind and tidal elevation data, form the base from which a design study can be carried out.

The model has been designed in such a way that the geometric features of a proposed layout can easily be read in, as described in Chapter 6. Furthermore, the program has been arranged so that design changes in the layout, or changes in the time interval, can be easily made without extensive revisions to the data.

Once set up, the model simulates the transport of a substance through the canal network, and writes and plots the results at user-specified time intervals. This means that the designer can follow the course of a flushing simulation to determine whether the rate of transport of the substance is sufficient to meet design criteria.

In summary, the numerical model, CANNET3D, developed to simulate mass-transport in low energy tidal finger canal networks, is a very useful design tool to enable the design engineer to evaluate and develop a canal network to conform to design and water quality criteria. In the following chapters of this report, the model will be used as the major component in a design procedure outlined to give the design engineer an approach to laying out a canal network that will make best use of the available energies of the wind and the tide for flushing. Once an initial design is formulated, the ability of the model to handle different pollutant load conditions is evaluated. Design changes are then made in a systematic manner based on the variability analysis to determine what features might be incorporated into the proposed system to improve its flushing action. In this iterative manner, an initial design layout can be taken, and with some alterations converted into an efficient system which will benefit the surrounding ecosystem, and the prospective residents.

CHAPTER 10

THE CANAL DESIGN PROBLEM

A canal design is a set of specifications describing the desired appearance and operational characteristics of the 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 between:

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.

10.1 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 10.1.

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 10.1. These terms will be more fully developed and illustrated in Chapters 11 and 12.

Design *constraints* are defined separately 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.

10.2 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 will be 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, but represents a saving to the developer if proper construction 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 less 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 10.2) and environmental checklists (Table 10.3). 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 to minimize future problems and adverse impacts.

10.3 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 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 10.4 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.

10.3.1 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 resources 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, Sep. 5, 1975, 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, Sep 5, 1975, 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

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, Sep 5, 1975, p. 41292]. In 1977 the Corps of Engineers redefined *navigable waters* (inasmuch as they apply to residential canals in the coastal zone) as "navigable waters of the United States":

> *Navigable waters of the United States* means those waters of the United States that are subject to the ebb and flow of the tide shoreward to the mean high water mark and/*or* are presently used, or have been used in the past, or may be susceptible to use to transport interstate or foreign commerce... Manmade 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, Jul 19, 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, Jul 19, 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, Jul 19, 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 10.5. The principal objective of these extensions and clarifications was to provide specific protection to "important resources" [Corps of Engineers, Fla. DER, Fla. DNR, 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, Jul 19, 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.
> [Bur. CZP, Oct 1977, p. 1]

The significant federal laws regulating canal development are summarized in Table 10.5.

10.3.2 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 and the Department of Natural Resources. 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 those capabilities is already existing.

10.3.2.1 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 [Bur. CZP, 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.

10.3.2.2 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" [Bur. CZP, 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, and that the one-hundred year hurricane may become a standard design storm.

10.3.2.3 State Legislation

The specific state legislation applicable to canal projects and related construction work is summarized in Table 10.6, taken from a more extensive table in the CMP workshop draft [Bur. CZP, pp. 66-67]. For projects not in the 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 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, 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, 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, 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 that the canal designer should be aware of is the classification of the waters of Florida in terms of usage (Chapter 17-3, 403 FS, Pollution of Waters). Class I 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 10.7 shows the criteria for Class III waters. The policy stated originally by the Department of Pollution Control, and adapted by the Department of Environmental Regulation,

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

10.3.3 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 10.8 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.

10.4 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, et al, 1977] is available from local permitting 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 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; the transportation/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 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 of 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, 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, then it will also require special management consideration from the state [Bur. CZP, "Florida CMP", 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, et al, 1977, p. 5].

Table 10.1 - Examples of Design Objectives, Guidelines, Criteria, and Constraints.

<u>Examples of Canal Design Objectives</u>

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

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

<u>Examples of Canal Design Guidelines</u>

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 go to detention ponds.

<u>Examples of Canal Design Criteria</u>

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

<u>Examples of Canal Design Constraints</u>

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 $\pm$ 10 degrees of prevailing summer wind direction.

Table 10.2

Table 10.2 - 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.

Table 10.2 - continued.

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 10.3 - 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 valve	○	●	●	●	●
Scenic areas and parks	●	●	●	○	●
Wilderness areas and forests	●	●	○	○	●
Historic sites or structures	●	●	○	○	●
Archaeological sites	●	●	○	○	●
F. LAND USE (Adjacent)					
Conservation and preservation areas	●	●	●		
Recreational	●	●	●		

Table 10.3 - continued.

	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	●	●	●	○	●
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 10.3 - continued.

	Site Selection and Evaluation	Site Planning	Building Design	Construction	Management
Employment				●	●
Location	●				
Accessibility	●				
Number of employees	●				
Unemployment	●				
L. SOCIAL REALITIES					
Population (Market Area)					
Total population	●	●	●		
Density	○	○	●		
Distribution	●	●	●		
Ethnic composition	●	●	●		

	Site Selection and Evaluation	Site Planning	Building Design	Construction	Management
Community structure					
Social structure of the community	●	●	●		
Relocation of people, buildings, services, etc.	●	●	○	○	○
Cultural attitudes	●	●	●		
Boundaries of political units or special districts	●	●	○		
Employment opportunity	●	●	●	●	○
Educational opportunity	●				
Physical, mental, and emotional health	●	●	●		
Personal safety	●	●	●	●	○

● primary importance

○ secondary importance

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

Table 10.4 - 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

ADAPTED FROM Coastal Coordinating Council, 1974, page 130.

<u>Source:</u> Veri, et al, 1975, p. 169.

Table 10.5 - 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 dredqed materials.
U.S. Army Corps of Engineers* - Federal Register 19JUL77	Regulatory program of C of E in "Waters of the United States".

**memo of understanding with Secretary of the Interior, reliances of U.S. vs. Holland*

Source: Snyder Oceanography Services, 1978.

Table 10.6 - Authorities to be Utilized and State Agencies Involved in Activities Related to Canal Development.

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

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

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 & Treatened Species Act
Ch. 193 F.S.: Tax Assessments
Ch. 418 F.S.: State/Local Recreation Coordination
Ch. 592 F.S.: Recreation & Parks
Ch. 177 F.S.: Coastal Mapping Act
Ch. 403. 165 F.S.: Pollution Recovery Trust Fund

Implementing Agencies:

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

Additional Authorities and Implementing Agencies if DRI Project:

Ch. 380 F.S.: Environmental Land & Water Management Act - DSP: Div. of State Planning
Ch. 478 F.S.: Land Sales - 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 if Development in ACSC:

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

Table 10.7 - 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 specificially 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."

Table 10.7 - continued.

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 10.8 - 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 & Registration	Planning Commissions Florida Division of Land Sales and Condominiums
( ) Construction Codes	Building Officials
( ) Permits for Septic Tanks	County Health Department Florida Department of Health and Rehabilitative Services
( ) Regulations for Private Wells	County Health Department Florida Department of Health and Rehabilitative Services Regional Water Management District
( ) Permits for Private Docks, Bulkheads, Other Structures in Waterways	Florida Department of Environmental Regulation Florida Department of Natural Resources Jacksonville District, U.S. Army Corps of Engineers
( ) Permits for Alteration of Wetlands	Florida Department of Environmental Regulation Florida Department of Natural Resources Regional Water Management District Florida Game & Freshwater Fish Commission

Table 10.8 - continued.

RULES AND REGULATIONS	AGENCY TO CONTACT FOR INFORMATION
( ) 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 11

CANAL DESIGN OBJECTIVES, GUIDELINES, CRITERIA AND CONSTRAINTS

Canal design objectives, as defined in Chapter 10, 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.

11.1 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 great deal of planning and insight. To do it properly, the planner cannot ignore any of the many factors which influence and constrain the problem: the natural characteristics of available sites, the land-use guidelines which have already been established in the area, the restrictions imposed by 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 11.1. Here the various considerations are categorized in accordance with the individual or group responsible for making decisions, and the 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 Figure 11.2 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 a very important decision because it determines, by means of the lot size, 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. A certain degree of future flexibility can be designed into this decision, however, if the lots are permitted as small as local zoning allows and then sold, at least initially, in multiples of two or more.

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 manuevering. 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, May, 1975, p.7].

The source of water supply must be from off the site unless saltwater intrusion is not a problem at the particular site, 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 watertable 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, May, 1975, 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, May 1975, 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, May 1975, 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, May 1975, pp. 178-179].

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

In general, it would seem reasonable to conclude that septic tank systems are preferable *provided* water and soil characteristics are right and *provided* absorption fields are located and built properly with adequate capacity. Onsite central treatment plants not only have a higher capital cost and 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 [May 1975, pp. 3, 9, 159-186], and the U. S. Public Health Service's *Manual of Septic Tank Practice*, [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 this can also be used as a park or recreation area during normal conditions. The alternative is to provide the necessary drainage systems, collection facilities, pipes and pumps for moving storm water offsite. Storm water and waste water systems can also be combined. althouoh this is an inefficient and costly wav to recycle water.

The advantages of slopinq canal banks over bulkheaded banks were briefly mentioned in Chapter 2. 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 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 less 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 nothing more than the first stage in the planning process, it is noted that in selecting and/or evaluating a site there are two important factors to consider: [Veri, et al, 1975, p. 130]

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

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 12 the assessment of site charecteristics will be considered.

11.2 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 attempt to find the depth 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 11.3. This table also shows the principal viewpoint of the institution or agency which produced the guidelines, and the table in which each is quoted or summarized.

The first of the six sets of guidelines (Table 11.3) 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 11.3. The complete text of all three responses will be found in Appendix H. The point made by Col. Goode of the Corps, that artifical waterways should more closely follow the examples designed by nature, is a good one, which 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 11.4) has been taken from the booklet accompanying the forms for joint permit application for dredge and fill projects in Florida [Corps of Engineers and State of Florida, 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, quite 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 can 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 11.5) 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 11.6) by 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 11.7) 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 (Figure 11.1) 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 Figure 1.5 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, development, conservation and planning viewpoints. After all, they are all trying to fulfill the same, interrelated objectives with the same natural resources.

Table 11.1 - 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 11.2 - Principal Design Objectives Relating Particularly to Residential Canal Design.

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

Table 11.3 - Decision-Making Agencies or Institutions with an Interest in Coastal Development.

Decision-Making Area	Agency or Institution	Viewpoint	Table or Figure
Federal Government	Environmental Protection Agency U.S. Fish & Wildlife Service	regulatory biological	Table 11.4
Federal and State Government	Corps of Engineers Florida Department of Natural Resources	regulatory regulatory	Table 11.5
Construction Industry	Associated General Contractors	development	Table 11.6
Conservation	The Conservation Foundation	conservation	Table 11.7
Regional Planners and State Government	Georgia Coastal Zone Management Technical Committee Georgia Department of Natural Resources	planning regulatory	Table 11.8
Private Industry	Canal Designer	rational engineering design	Figure 11.1

Table 11.4 - 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.N., 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."

Table 11.4 - continued.

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 11.4 - 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."

Table 11.4 - continued.

f. <u>septic tanks</u>: "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. <u>entrances</u>: "Placement of canal openings where boat traffic to and from the canals will cause erosion of adjacent or opposite natural shorelines."

h. <u>dredging into the aquifer</u>: "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. <u>dredge spoil</u>: "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. "<u>Hydrology of Site and Surrounding Area</u>"

a. <u>channel routing</u>: "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. <u>tidal differential</u>: "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 11.4 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."

Table 11.4 - continued.

"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 11.5 Design Guidelines for Dredge-Fill Structures, 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."

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

Table 11.6 - Design Guidelines Relative to Canal Design for Various Kinds of Ecosystems in South Florida, Offered 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."

Table 11.6 - continued.

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

<u>Constraints</u>

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 11.6 - 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."

<u>Opportunities</u>

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."

Table 11.6 - continued.

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 11.6 - 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

<u>Constraints</u>

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."

Table 11.6 - continued.

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 contaiminate 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."

<u>Opportunities</u>

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 11.6 - 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 biological 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.

Table 11.6 - continued.

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 11.6 - 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."

Table 11.6 - continued.

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 11.7 - 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."

Table 11.7 - continued.

Applicable Element	Specific Guidelines	Implementation
"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 11.7 - 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].

Table 11.7 - continued.

Applicable Element	Specific Guidelines	Implementation
"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 11.7 - 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 readways 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."

Table 11.7 - continued.

Applicable Element	Specific Guidelines	Implementation
"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 sharacteristics and soil permeability."

Table 11.7 - 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 whereever 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 11.8 - Design Guidelines Relative to Canal Design Offered by Planners and State Agencies.

VEGETATION...	
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.

Table 11.8 - continued.

SOILS...

Not Considered	Considered
"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 11.8 - continued.

Not Considered	Considered
NATURAL DRAINAGE (cont'd)...	
"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.

Table 11.8 - continued.

Not Considered	Considered
"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 <u>with</u> the land."

Table 11.8 - continued.

PLANNING FOR CONSTRUCTION: SITE CHARACTERISTICS...	
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. Once this problem 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.

Table 11.8 - continued.

SITE (cont'd)...

Not Considered	Considered
"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 contruction.	"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 branches will help to offset any root loss.

Table 11.8 - continued.

SITE (cont'd)...

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.

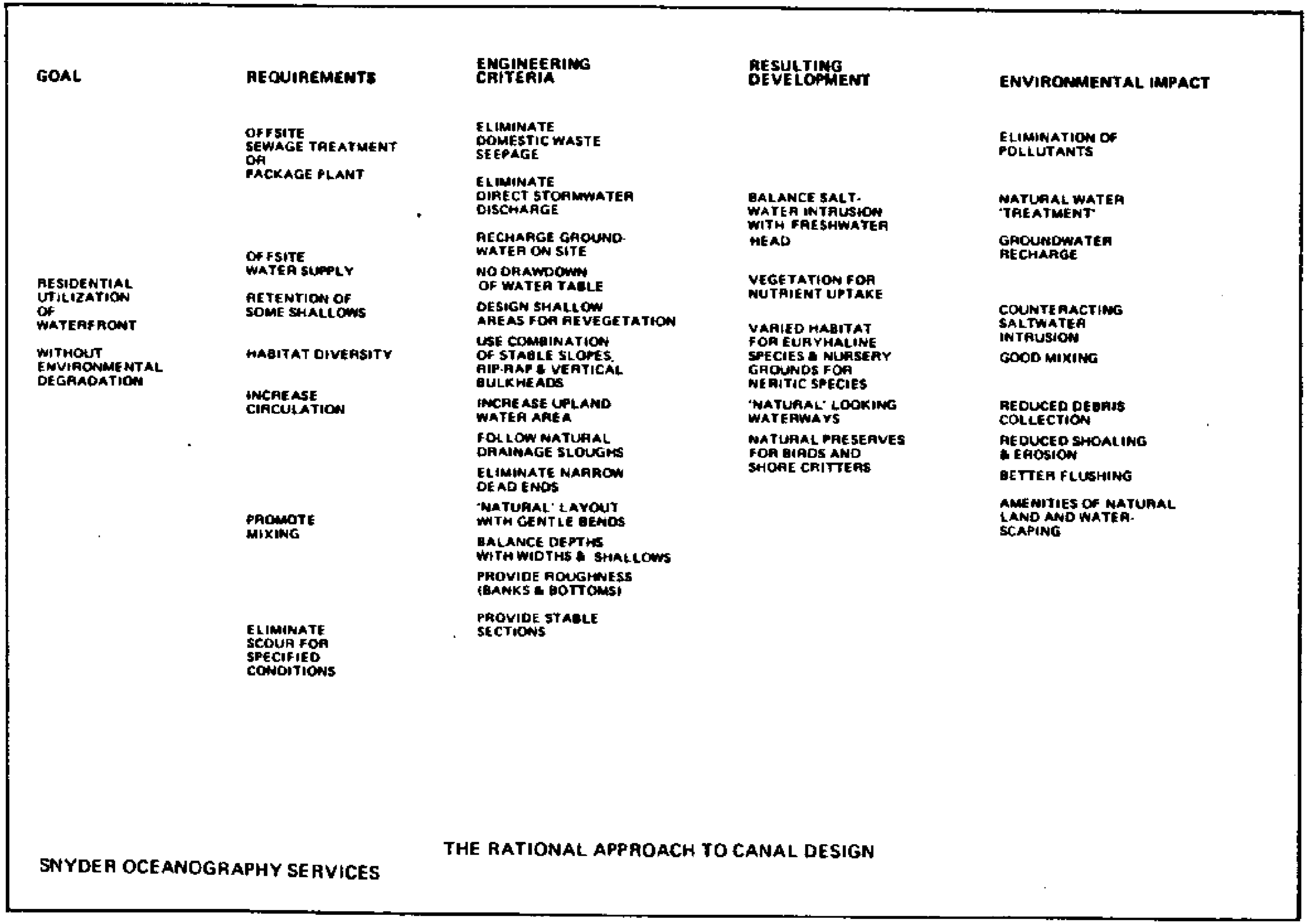


Figure 11.1 - The Rational Approach to Canal Design
(Snyder, R.M., *Residential Canals and Environment*, Snyder Oceanography Services, Jupiter, Florida, 1976).

CHAPTER 12

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 12.1, 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), aquatic life, 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 always 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 characteristics 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.

12.1 Fixed Characteristics

12.1.1 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 usually provide better views. Some variation in topography is necessary for handling storm water and for protecting homesites from standing water. If the site is flat, some changes in topography may be required to control drainage.

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.

Aerial photography is often a useful method for obtaining topographic data, especially if the photographs are taken in stereo pairs with suitable ground control. The interpretation of stereo aerial photography 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 in = 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.

Maps and aerial photographs are usually available from local or county government agencies, planning groups, and local libraries and archives. Some specific sources are listed in Section 12.3.1. 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.

12.1.2 Tidal Range

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

The determination of the legal boundary of a property with a tidal shoreline is indirectly related to the tidal range. 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 in 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 Section 12.3.2.

12.1.3 Climate

Data on wind, precipitation, and other climatic variables are available for selected areas in all states. Figure 12.1 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 12.1, 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. Dept. of Agriculture (see Section 12.3.3.2.B1). 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 Dept. 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 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 quite 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 Section 12.3.3.

12.1.4 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 geologic and hydrologic background, 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 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, and try to see whether they have any visible connection with an existing canal or the receiving waterbody. He should try to determine, from local knowledge, whether such waterbodies are temporary or permanent, and 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 its scenic quality, note the surrounding types of vegetation and qualitatively evaluate its recreational value or wildlife habitat potential. It is also important to consider what the effect of construction activity on such a waterbody could be.

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

The boundaries of the property should be investigated to determine whether there are any sources of water from neighboring properties which could affect the quantity or quality of water on the site. 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.

Groundwater is 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 principal concern is not to cut canals into the aquifer or below the water table. If no test wells are available, it would be advisable to employ a competant hydrologist to direct the tests needed to locate the water table and the aquifer, and to determine infiltration rates at potential detention basin sites.

If water is to be provided for the community from underground supplies, a hydrologic survey by a competant 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 Section 12.3.4.

12.1.5 Vegetation

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

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

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

Some sources of information on vegetation are listed in Section 12.3.5.

12.1.6 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 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 with him a soils expert. 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 from 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 which 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 information on soils are listed in Section 12.3.6.

12.2 Alterable Characteristics

12.2.1 Drainage

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

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

During the preliminary site investigation, the canal designer must be alert for opportunities to improve various characteristics of the site. Particularly evident will be areas that do not have adequate drainage, such as depressions with highly impermeable soils that can flood and run off into the receiving waters or existing channels. The designer's overall viewpoint should be to preserve high ground for housing and to commit low ground for channels, and to consider whether the natural drainage is satisfactory. A prepared checklist of questions for identifying soil erosion problems similar to the following 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 which 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 Section 12.3.7.

12.2.2 Pollution Sources

Potential sources of pollution, which will be characterized in Section 12.5, are highly site-specific. It is the responsibility of the canal designer to take the necessary steps to locate and identify as many of the existing sources as possible so that they can be cited in permit applications, and so that they can either be eliminated or accommodated in the design.

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

12.3 Sources of Additional Information

12.3.1 Information on Maps and Charts

A. National Ocean Survey. "Nautical Chart Catalog." Available from NOS and its agents.

This catalog shows areas covered by small craft, harbor, coastal, and intracoastal waterway charts.

B. USGS. "Index to Topographic Maps of Florida." Available from local agents or Branch of Distribution, USGS, 1200 South Eads Street, Arlington, VA, 22202.

Index shows locations of 7-1/2-minute (1:24,000) and 15-minute (1:62,500) topographic maps on outline of the state.

C. USGS. "Publications of the Geological Survey 1962-1970." U. S. Government Printing Office, Washington, 1972. Monthly and annual supplements.

D. Bureau of Geology. "List of Publications." Info. Circular No. 87, Dept. Nat. Res., Tallahassee, 1975.

Contains listings of special-purpose maps, many related to water resources.

E. Resource Planning Section. "Handbook: Building in the Coastal Environment." may be obtained for $2.00 from:

Graphics Dept.
Coastal Area Planning & Development Commission
P.O. Box 1316
Brunswick, GA 31520

F. Agricultural Stabilization and Conservation Service Maps, information about ASCS coverage available, and requests for photo index sheets, may be obtained from:

Eastern Laboratory
Aerial Photography Division, ASCS
U. S. Dept. of Agriculture
45 South French Broad Avenue
Asheville, NC 28801

G. Information about Soil Conservation Service photo index sheets, aerial photographs, and mosaics may be obtained from:

Cartographic Unit
Soil Conservation Service
Federal Center Building
Hyattsville, MD 20781

12.3.2 Information on Tides and Boundaries

A. National Ocean Survey. *Tide Tables. High and Low Water Predictions, East Coast of North and South America.* Annually, NOAA, Washington, DC, from NOS and its agents.

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

C. U. S. Coast and Geodetic Survey *Tidal Beach Mark Indexes, Parts I, II, and III.* Washington, DC, 1969.

D. Predictions Branch
Oceanographic Division
National Ocean Survey
NOAA, U. S. Dept. of Commerce
Rockville, MD 20852

Provides listings of tidal harmonic constants for reference stations and subordinate stations listed in the NOS Tide Tables.

E. Hershman, M. J. "Boundaries, Ownership, and Jurisdictional Limits in the Coastal Zone", Section 5, Chap. 7, in Clark, 1977, pp. 581-587.

12.3.3 Information on Climate and Meteorology

12.3.3.1 General Information

A. The National Climatic Center provides on a subscription basis, for each of the various specified meteorological stations in a state:

1. "Local Climatological Data"

   monthly and annual summaries of:

   a) average daily:

      Temperature: maximum, minimum, average, departure from normal, average dew point, degree days (all °F)
      Weather type
      Precipitation (inches)
      Atmospheric pressure
      Wind: resultant direction, resultant speed, average speed, fastest mile (all mph)
      Sunshine: hours and percent of possible
      Sky cover: sunrise to sunset, midnight to midnight
      Sums and averages of the above.

   b) hourly precipitation.

   c) 3-hourly observation of:
      sky cover, ceiling, visibility, weather, temperature, relative humidity, and wind.

2. Daily Weather Maps:

   National Climatic Center
   Federal Building
   Asheville, NC 2880.

B. Water Information Center, Inc. *Climates of the States,* Two volumes, Port Washington, NY, 1974.

   Contains summaries (by state) on:

1. General climatic features

2. Temperature

3. Precipitation

4. Droughts

5. Snow

6. Wind

7. Tropical storms

8. Other climatic elements.

C. Environmental Data Service. "Monthly Normals of Temperature, Precipitation, and Heating and Cooling Degree Days." No. 81 (by states), NOAA, Washington, DC, Aug., 1973.

D. Environmental Data Service. "Daily Normals of Temperature and Heating and Cooling Degree Days." No. 84, NOAA, Washington, DC, Sep., 1973.

E. U. S. Weather Bureau. "Decennial Census of the United States Climate: Summary of Hourly Observations 1951-1960", *Climatography of the United States*, No. 82-1-82-48 (by cities).

Provides monthly and annual frequencies of occurrence of various values of temperature, precipitation, relative humidity, wind speed and direction, and cloudiness for over 250 cities.

F. Environmental Data Service. "Selective Guide to Climatic Data Sources", Key Meteorol. Rec. Docum. No. 4.11, 1969.

12.3.3.2 Additional Information on Wind and Precipitation

A. Wind:

1. Hourly wind readings are submitted by each station to the National Climatic Center as part of "Surface Weather Observations", available on a daily basis.

2. "Uniform Summary of Surface Weather Observations" by station for a specified number of years.

Gives percentage of wind direction and speed in eleven speed categories and sixteen directions, by month, with summaries.

B. Precipitation:

1. Soil Conservation Service. "Rainfall Frequency Atlas, Alabama, Florida, Georgia, and South Carolina, of Durations from 30 Minutes to 24 Hours and Return Periods

from 1 to 100 Years". U. S. Dept. of Agriculture. Reprinted from Weather Bureau Technical Paper No. 40, 1961.

2. Florida State Road Dept. *Drainage Manual.*

   Provides rainfall intensity-duration curves for zones in Florida.

3. National Climatic Center. "Climatological Data. Florida", monthly subscription, Asheville, NC.

   Provides monthly summaries of temperature and precipitation, with station list and map.

12.3.4 Information on Water Data, Water Supply, and Waste Water Disposal

12.3.4.1 Water Data

A. Office of Water Data Coordination. *Catalog of Information on Water Data.* Station Listings for each Water Resources Region:

Part A - Streamflow and Stage
Part B - Quality of Surfacewater
Part C - Quality of Groundwater

U. S. Geological Survey. Washington, DC 20244.

Provides listings for each of eighteen water resources regions in the U. S., giving for each station its location, period of record, form of data storage, drainage area, type of measurement, and type of data. The latter category specifies the availability of daily discharge, flood frequencies, roughness coefficients, flood plain maps, precipitation, tides, datum, sedimentation studies, and surface inflows-outflows data.

B. Maps showing the locations of stations listed in A. above are also available, generally at a scale of 1:1,000,000.

C. Office of Water Data Coordination. *Index to Areal Investigations and Miscellaneous Water Data Activities.* U. S. Geological Survey, Washington, DC 20244, 1970.

Lists reports on investigations by Federal agencies, non-Federal agencies, and USGS, with indices by state and subject.

D. Office of Water Data Coordination. *Recommended Methods for Water Data Acquisition.* Preliminary Report. Federal Interagency Work Group on Designation of Standards for Water Data Acquisition, USGS, Washington, DC, Dec., 1972.

Contents: Surfacewater, groundwater, fluvial sediment, biological, chemical and automatic monitors.

E. Bureau of Geology. "List of Publications." Information Circular No. 87. (Rev. 1975), Dept. of Nat. Res., Tallahassee, FL, 1975.

The Bureau of Geology publishes bulletins, information circulars, special purpose maps, reports of investigations and special publications describing geologic and hydrologic (surface and groundwater resources) for specific areas of Florida. Of particular interest for water resources are the reports of investigations for the counties, which provide historical water data (quantity, quality, and usage) as well as general information on the data collected, geologic setting, physiography, rainfall and evapotranspiration, and the hydrologic setting of each region.

12.3.4.2 Water Supply and Waste Water Disposal

N.A.H.B. *Land Development Manual.*

A handbook which covers planning and development regulations, environmental design, grading and erosion control, water supply and sanitary sewerage systems, and storm drainage and sewerage, among other topics. Available from:

National Assoc. of Home Builders
1625 L St., N.W.
Washington, DC 20036
Price: $7.00

12.3.5 Information on Vegetation

A. Gallagher, J. L., "Zonation of Wetlands Vegetation", in Clark, 1977, pp. 752-758.

B. "Indicator Plant Species of Coastal Wetlands". [Clark, 1977, pp. 894-905].

B. Carlton, J. M., "A Guide to Common Florida Salt Marsh and Mangrove Vegetation", No. 6, Marine Research Laboratory, Fla. Dept. of Nat. Res., March, 1975.

D. Snyder, R. M.: Section F, Appendix J of this report.

12.3.6 Information on Soils

A. Soil Conservation Service. "Soil Survey, ________ County, Florida", A series issued by U. S. Dept. of Agriculture in cooperation with U. of Florida Agricult. Exper. Station, Washington, DC, U. S. Gov't. Printing Office. Dates vary.

B. Wiseman, J. R. "Consolidation of Fine-Grained Soils", in Clark, 1977, pp. 587-589.

C. Reimold, R. J. "Constructive Use of Dredge Spoil", in Clark, 1977, pp. 589-593.

D. Clark, 1977, Appendix II, pp. 810-812.
   a. soil texture and structure
   b. soil texture identification

12.3.7 Information on Drainage

A. Effects of Altering Natural Patterns of Land Drainage [Clark, 1977, p. 547].

B. "Land Drainage" [Clark, 1977, pp. 397-404].

C. "Preservation of Natural Drainage" [Clark, 1977, pp. 538-543].

D. N.H.B.A. *Land Development Manual.* (See Section 12.3.4.2)

12.4 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.

12.4.1 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 investigation are:

1. To determine the conditions which will limit field measurements at the site, such as overall depth of receiving waterbody, prevailing currents, proximity of dwellings, intensity of boat traffic, access to site from both land and water, and security at site.
2. To determine if any unusual conditions are present which will require additional measurements, special equipment, or special procedures.
3. To determine the approximate range for the variables to be measured (which determines the type of instrumentation and support equipment that will be required).
4. To locate elevation reference points for future surveying.
5. To familiarize field crews with site logistics.
6. To measure depths in canals for later determination of support equipment requirements.

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

1. Support the design analysis for the preparation of the trial canal system designs.
2. Enable estimates to be made of calibrating coefficients for the numerical model.
3. In this case of development of a numerical model, to obtain suitable data for model verification.

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

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

12.4.2 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 and perhaps dye concentration, 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. 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, and taken 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 boat with a complete set of equipment and at least three personnel. 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 X 3 surface locations, X 6 depth locations, or 198 sampling points. Assume that the boat requires ten 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 ten minutes. It would still take five and one-half hours (slightly less than one-half tidal cycle) to complete the measurements at all thirty-three locations. Such an approach certainly would not provide useful data, as there is too great a time span involved.

Another example will demonstrate the practical limits to sampling. 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 just under an hour. These measurements could not be made and recorded by hand, at a rate of approximately three per minute, but could be sampled and recorded at this 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 useful synoptic measurements are required over a several-day period.

When planning either a preliminary site investigation or a field survey, it is advisable to consider subcontracting field work to an organization which has had experience in coastal oceanographic and canal hydraulic measurements. The canal designer should satisfy himself that this organization has the proper instrumentation and equipment, and 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 detailed 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 in the long run since the reputable leasing companies maintain the equipment and it can be transported efficiently by air freight. Unfortunately, specialized instruments such as electromagnetic current meters and fluorometers are not ordinarily available from leasing companies.

12.4.3 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 12.2 relates the desired information, information derived by engineering analysis and measured variables involved in canal design.

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

Tidal elevation should be continuously recorded during the field survey. These data are needed for referencing channel depths, current velocity readings, and salinity, water temperature and water quality measurements taken either from a boat or from the shore line. 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 that in general, in Floridian canals, such uniform conditions do not very often prevail.

If it is determined that wind velocity has a significant effect on the circulation it is necessary, for verifying numerical model results, to have a continuous recording of the wind. The wind sensor must be placed at a height sufficiently 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 m.

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, SCS) 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 to supplement existing 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 jurisdiction which 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 pollution 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 probability of erosion or deposition.

12.4.4 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,

1. Accuracy: 1 to 5 percent full-scale is adequate for most measurements. Usually instruments with analog meters for readouts will have no better than 2 percent accuracy. Digital readouts are preferable provided they can be easily read in bright sunlight.
2. Reliability: Best available is required. The high cost of a field survey requires that all equipment be operational for the entire period of the survey.
3. Power source: 12 volts DC, with sufficient regulation and protection for operation from lead-acid batteries, or as a second alternative, operation from built-in rechargable cells with low power consumption is required. Operation on 12 VDC is preferable if the equipment is to be used in a boat which has an alternator on the engine, since then recharging is relatively convenient during and at the end of the work day. If only 110 VAC operation is possible, an inverter is preferable over a generator because the noise and exhaust from the latter are difficult to work with and may adversely affect the crew's performance.
4. Environmental conditions: Salt moisture, direct sunlight, rain squalls and relative humidity approaching 100 percent at night are some of the conditions that must be planned

for. Equipment with mechanical parts require special precautions to protect against sand and corrosive moisture. Equipment should be relatively light and portable since often it must be carried to the site.

As an illustration of the importance of environmental conditions, the Hydraulic Laboratory lost an entire set of current meter readouts 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.

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

6. Calibration: It is desirable that simple, direct means of calibration either be built into each piece of equipment, or made available in the support equipment. This assures more reliable operation, and usually serves as a check on whether the equipment is functioning properly or not.
7. Recording: Where possible, arrangements should be made for recording data against a reliable time base, with some means available for recording notes by the operator at the time measurements are being made. This is particularly true when measurements are taken from a moving boat. Automatic locating and recording systems for boat position are not usually available for survey boats, so manual recording of navigational position on a strip chart is often the only practical solution.

12.4.5 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 12.3 summarizes the types of equipment and instrumentation which can be used for each of the measured variables listed in Table 12.2.

12.4.5.1 Tide Recording

A reliable tide recorder is essential for canal survey work. A number of different water level recorders are available and suitable for this application. Recording is available in a variety of formats, either continuous or discrete in time. A recorder with a spring-driven clock and fully-mechanical mechanism has the advantage of not requiring electrical power, but it still does require replenishment of the recording media at regular intervals. The Hydraulic Laboratory has found the Leupold Stevens Type F, Model 68, to be extremely reliable over long periods of time. This instrument features a choice of elevation and time ranges, by means of replaceable gears, and an accurate eight-day spring-driven clock. Thus no external power is required. A recording tide gauge will require a stilling well for dampening wave action, installing the recorder, and providing security for the equipment. Actual techniques for installing the equipment are dependent on the physical arrangement of the stilling well and depend to a great extent on the length of time the gauge will be installed. The structural requirements should be determined for the length of time the gauge is to be installed, to withstand expected climatic conditons and natural and man-caused interference, and to provide security for the equipment.

The recorder is mounted on a stilling well in a box which can be locked, as illustrated in Figure 12.2. 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 either be mounted on a platform, such as a dock or bulkhead, or supported in a free-standing manner 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 effects of wakes. 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 which supports a permit application must be referenced to MLW, and such a reference would be required if the data were ever contested in legal proceedings. The only time the tide record need not be referenced is for purposes of a preliminary site evaluation, in which the approximate tidal range at the site is being confirmed. It is not possible to determine MLW from an unreferenced record to any acceptable degree of accuracy because the tidal elevations are subject to a variety of offsetting factors, including bottom topography and local storms.

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

12.4.5.2 Distance Measurements

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

12.4.5.3 Depth Recording

A Benmar echo sounding recorder, Model DR-68, is used for taking depth profiles. This 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 the water. The instrument is powered by 110 VAC, so an inverter has been provided to convert the 12 VDC boat supply for this purpose. The recorder 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 other reflections. The instrument has an absolute accuracy within $\pm$ 0.1 ft in the first 10 ft of depth, as verified in the field.

12.4.5.4 Current Measurements

At the beginning of the canal project an Ott laboratory propeller meter, Model C.1, was used to record 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, it is a pulse counting system which requires that the reading be integrated over at least a minute, and often several minutes if significant differences in successive pulse counts are obtained.

After experiencing difficulties measuring very low currents, a Savonius-rotor current meter (Savonius, 1931) 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 had no directional capability.

After experiences with inadequate instrumentation, a Cushing dual-axis electromagnetic current meter, Model 82-CP velmeter sensor with Model 632-P portable converter, was purchased for the project and provided the necessary low range and directional capability. 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 in two layers are a common part of the hydrodynamic make-up of tidal canals, and that even three-layer flows exist. This meter consists of a probe, 3/4 in in diameter and 11 in overall, connected by cable to a signal converter mounted in a hermetically sealed, internal-battery-powered, meter box. The probe measures the instantaneous longitudinal and lateral velocity components at a point 3 in 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 scale can be changed to as low as 0.3 fps, with graduations at 0.01 fps. The output time constant (integrating time) associated with a reading can be changed in discrete steps of 0.1 to 10 seconds. The manufacturer specifies the linearity of the system to be $\pm$ 1 percent of full scale, the zero offset to be $\pm$ 0.01 fps or better, and random noise (rms) to be $0.002/\sqrt{T}$ fps, where T is the output time constant in seconds. For operation with a time constant of one second, this is an overall accuracy of ± 0.012 fps or better. The threshold of this meter is about 0.01 fps and the overall accuracy, using the panel meters, is about ± 2 percent.

The probe is extremely sensitive to any movement. A portable tower was therefore designed and built (Figures 12.3a and b) 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 shore (Figure 12.4). A carriage holding the velocity probe is free to slide up and down the tower, enabling the probe 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. The monitoring is performed on the bank of the canal or on the work boat. Two output signals for each probe, each with an output of ± 5 VDC, 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 profile has been recorded, the probe can be repositioned at the depth of the average current and 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 12.5), 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 Hydraulic Laboratory's 80 ft flume. 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 $\pm$ 0.05 to $\pm$ 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 $\pm$ 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 $\pm$ 1 percent, were plotted against the speed of the towing carriage. A 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.

12.4.5.5 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 only be held about 10 ft off the surface of the water. A measurement taken this close to the water is a measurement of a local wind component which has been affected by the presence of trees and structures in the vicinity of the site, and is therefore not representative of the wind system prevailing over the whole canal. In addition, the hand-held meter is not directional, and it cannot provide the long-period recordings (four to five days continuous) needed to evaluate the effect of winds on the circulation.

An R. M. Young windvane and three-cup anemometer, Model 6001, with a Model 6420J recorder/translator, is used for long-term wind recordings. Mounted on a ten-meter-high telescoping antenna mast, the sensor is well above the tops of most trees that grow along canal banks. The mast can be set up by two persons in about forty-five minutes, and a hand-held compass 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 to be better than 5 percent (2.5 mph). The meter was calibrated in the factory before delivery and used immediatly upon receipt for the October 1977 field trip to the 57 Acres site, so no additional calibration has been performed by the Hydraulics Laboratory.

12.4.5.6 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 operation it was disconnected and replaced by an external supply 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 $\pm$ 1 ppt.

Salinity is not measured directly by conductivity measurements. The salinity of seawater is approximately the mass ratio of total dissolved solids to the total sample of water. Because the composition of pure seawater is almost completely uniform throughout the oceans, the salinity of pure seawater can be expressed in terms of the conductivity of the ions in solution and water temperature (Figure 12.6). 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 a 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],

$$\text{Salinity} = 0.03 + 1.805\,(\text{chlorinity}) \tag{12.1}$$

which is incorporated in a correction factor (due to Harvey [1963, in LaMotte Chemical Products Co., undated]) provided with the test kit. Salinity and temperature may be converted to density (seawater only) by means of the *Handbook of Oceanographic Tables* [U.S. Naval Oceanographic Office, 1966].

In general, the conductivity method is faster and more convenient for salinity measurement than the titration method, even though a simultaneous temperature measurement and a table are required for conversion to salinity from conductivity. The principal advantage is that it is far easier to lower the conductivity and temperature probes to the desired depth than it is to bring up a water sample, pour it off into a sample bottle and then perform the various steps of the titration. Better conductivity instruments are readily available, some with automatic temperature correction, but these should always be supplemented by as many titrations as possible to ensure 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.

12.4.5.7 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. 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 or 0.07 percent at a salinity of 10 ppt and a temperature of about 30°C) in Floridian canals.

12.4.5.8 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 electronic instrument has certain advantages over titration and, once mastered, provides more measurements per unit time. Only limited numbers of dissolved oxygen measurements have been conducted by Hydraulic Laboratory field crews, almost all by titration, so no comparison of results would be very 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.

12.4.5.9 Dye Concentration

The Hydraulic Laboratory uses a Turner Designs Model 10-005 field fluorometer for measurement of the concentration of the fluorescent water-tracing dye Rhodamine WT. The operation of a fluorometer is based on the principle that a fluorescent material has the ability to absorb light at one wavelength and respond almost instantly with the emission of light at a new and slightly longer wavelength. The intensity of the emitted light is proportional to the concentration of the fluorescent material present in the sample. The fluorometer, making use of this property, irradiates the sample through a filter designed for the particular type of dye being 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 which 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 $\pm$ 1 percent and its resolution is to $\pm$ 0.5 percent of full scale. The instrument provides analog outputs proportional to the reading of concentration and the range setting, and an output representing the setting of the range multiplier, suitable either for recording on a strip chart recorder or in a data acquisition system. One particularly useful feature is the automatic range change over two ranges, 10^{-7} to 10^{-9} and 10^{-9} to 10^{-11}. The change of range from one of these two automatic ranges to the other is accomplished by means of a manual switch. The fluorometer can be arranged either to measure the concentration of a sample in a cuvette, or a sample pumped continuously through the machine. The fluorometer has been used successfully in both modes in the field.

When used at a station to measure the concentration of dye at various depths, it is sometimes convenient to set the instrument up for individual samples which have been taken in water sampling bottles. This is particularly true when other measurements are to be obtained from the water samples at the same time, such as salinity, temperature, or dissolved oxygen. More often, however, a continuous sample is desired at a specific location. Such a sample can be pumped up a hose and through the fluorometer with the continuous sampling cuvette installed. Several electronic instruments for other measurements can 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 12.7. 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 vinyl tubing, which tends to absorb minute quantities of dye, and stresses that the hose or tubing must be completely opaque for a distance of at least 2 ft on both sides of the fluorometer. It has been found that rubber garden hose is adequate for this application, although there can be some difficulty in maintaining air-tight connections. Higher quality commercial hose and quick-connect couplings are now under 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 simply putting 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 so that water is entrained from the desired depth in a horizontal plane. If any debris is known 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 12.8. 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 12.9). 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 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 which are opaque to sunlight (to minimize photodecomposition), one with 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 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, and then the standardization process is considerably simpler.

12.4.6 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 which 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 data and the information gained from the analysis.

12.4.6.1 Data Reduction

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.

12.4.6.2 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 which 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 $\pm$ 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 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.

Tidal elevations are most convenient for the designer in tabular form. These listings of tidal elevations are not included in the report, but are on file at the Hydraulics Laboratory. The depths in the canal should be represented both by longitudinal and lateral profiles similar to those in Figure 2.12.

12.4.6.3 Reduction of Current Readings and Presentation

Readings of longitudinal and transverse velocity components may either be recorded on a dual-channel strip-chart recorder or logged.

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 Appendix A, where the June, 1977, Loxahatchee north canal data are presented (for example, Figure 12.10). 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 Section 13.4.3.

12.4.6.4 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. An example of the former is shown in Figure 2.35. For verification of the numerical model on an existing canal system, the half-hourly wind data are punched directly on cards for input to the numerical model.

12.4.6.5 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 and are additionally somewhat inconvenient in that, to obtain values to the nearest 0.1 ppt, a three-way interpolation within the table must be performed. Accordingly, in order to be able to reduce conductivity measurements efficiently, and to process data for temperatures above 30°C, the computer program presented in Appendix C was written.

The average density of seawater varies between 1.022 and 1.028 g/cm^3. It is a function of temperature, salinity and pressure, which are readily converted to density by means of tables referenced in Section 12.4.5.6. It is often convenient to express the density in terms of the oceanographic variable "sigma-t"

$$\sigma_t = (\rho - 1)1{,}000 \tag{12.2}$$

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 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 2.25 is an example of one of these comparative graphs.

12.4.6.6 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. 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, 1975]. It is sold only in solution form, and it has been approved for use as an alternative to Rhodamine B by the U.S. Public Health Service [1966, in 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 fluorescent 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, May 1975], no specific tests 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 is a function of the incident solar radiation over the range of photochemical response of the dye. 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 [May 1975, p. 210] attempted to measure the decay coefficient during one of their canal dye studies in September, 1974. For this experiment clear glass bottles containing known concentrations of Rhodamine WT were suspended at mid-depth, and measurements of sunlight intensity were recorded at the surface. 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 Rhodamine WT)" EPA [May 1975, p. 210]. Figure 12.13 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.

The unit of solar energy used by EPA [May 1975] is microeinsteins per square meter, corresponding to the scale on a pyrheliometer which was used for the measurements on site. 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

$$F = \frac{\text{velocity of light}}{\text{wavelength}} \tag{12.3}$$

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

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

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

$$E_\gamma = hf \tag{12.4}$$

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

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

where h = Planck's constant, [Js]

Thus, one einstein is equivalent to

$$Ei = N_A E_\gamma \tag{12.5}$$

$$= 6.02252 \times 10^{23} \times 3.369 \times 10^{-19} \text{ J}$$

$$= 2.029 \times 10^5 \text{ J}$$

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)

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

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

or $$= 4.8472 \times 10^{-6}\ ly \text{ at } 590\ nm \tag{12.6}$$

This conversion factor was confirmed by EPA by personal communication in March 1978. However, it is recognized that this conversion factor has only been calculated for one wavelength component of the solar radiation. A conversion factor for the total daily solar radiation incident on the water surface at a specific location would require a double integration over the range of wavelengths which cause photodecomposition of Rhodamine WT, and time. Such a calculation would have to be made knowing the characteristics of the atmospheric attenuation of solar energy for the particular day of interest, which is caused by scattering and absorption of gas molecules of pure dry air, water vapor, and dust particles [Bolsenga, 1964, p. 1]. The solar energy intensity affecting the dye at various depths in the water would have to then be determined from the extinction spectrum at various locations in the canal, which will depend on the characteristics of the water and it turbidity.

For the present, since the order of magnitude of measured photochemical decay at mid-depth in the North Carolina canals has been found to be significant, it is recommended that for any dye dispersion experiment which is to be conducted over a period in excess of one day, a sample of the dye should 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 recommended, therefore, 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 will be automatically taken into account.

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

$$\frac{c}{c_o} = \exp(-0.027t) \qquad (12.7)$$

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 profiles (for example,

Figure 2.23),as vertically averaged profiles (Figure 12.14), or as continuous profiles for each reach as in Figure 2.29, depending on whether they are point measurements or continuous measurements. The latter plots may be obtained by using the computer program documented in Appendix C.

12.5 Sources of Pollution in Canal Systems

In recent years the term "pollution" has often been used, and misused, for almost any observed consitutent 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 which 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 10.3.2.3).

12.5.1 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 12.4 summarizes the principal pollution sources and the types of consitutents which 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 12.5 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 which occur as a result of other than hydrodynamic forcing functions.

12.5.2 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 of population density in terms of type of dwellings and lot size is given in Table 12.6, and a set of average residential water use and sewage production data are shown in Table 12.7. 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 which 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 12.7. Goodman and Foster [1969 in Eckenfelder, 1970, p. 60] give lower total volumes of sewage production than in Table 12.7, varying from 70 to 100 gal/c-d depending on the type of housing (Table 12.8). 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 12.9. 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 12.10.

12.5.3 Septic Tanks

The principal factor governing the effectiveness of septic tank disposal systems is, in most instances, the characteristics of the soil in the vicinity of the absorption field [EPA, May 1975, 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, May 1975, 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, May 1975, 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, May 1975, 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, May 1975, p. 177]. Reports of distances traveled by coliform bacteria range from 10 to 2,000 ft [EPA, May 1975, 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 was 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

$$\begin{aligned} q_I &= 90 \frac{\text{gal}}{\text{cap-day}} \times 3 \text{ cap/lot} \times \frac{1}{7.481} \frac{\text{ft}^2}{\text{gal}} \\ &= 36 \text{ ft}^3\text{/lot-day} \\ &\simeq 40 \text{ ft}^3\text{/100 ft lot-day} \\ &\simeq 0.4 \text{ ft}^3\text{/ft canal length-day} \end{aligned}$$

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

$$\begin{aligned} q_I &\simeq 2 \times 0.4 \frac{\text{ft}^3}{\text{ft-day}} \times \frac{1}{24} \frac{\text{day}}{\text{hr}} \\ &\simeq 0.03 \frac{\text{ft}^3}{\text{ft-hr}} \end{aligned}$$

The concentration of a typical residential BOD coefficient is estimated to be on the order of 90.7 g/cap-day (Table 12.8) or

$$c_I \doteq \frac{90.7\ \text{g/cap-day}}{90\ \frac{\text{gal}}{\text{cap-day}} \times 3.785\ \frac{\ell}{\text{gal}}}$$

$$\doteq 0.267\ \text{g}/\ell \text{ or } 267 \text{ ppm}$$

12.5.4 Boats and Marinas

The effects of operation of outboard engines on the quality of water and the life systems in a canal would be negligable even if the canal were completely filled with boats, according to information from EPA-funded research [Appendix J, Section J, EPA, 1974b]. 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.

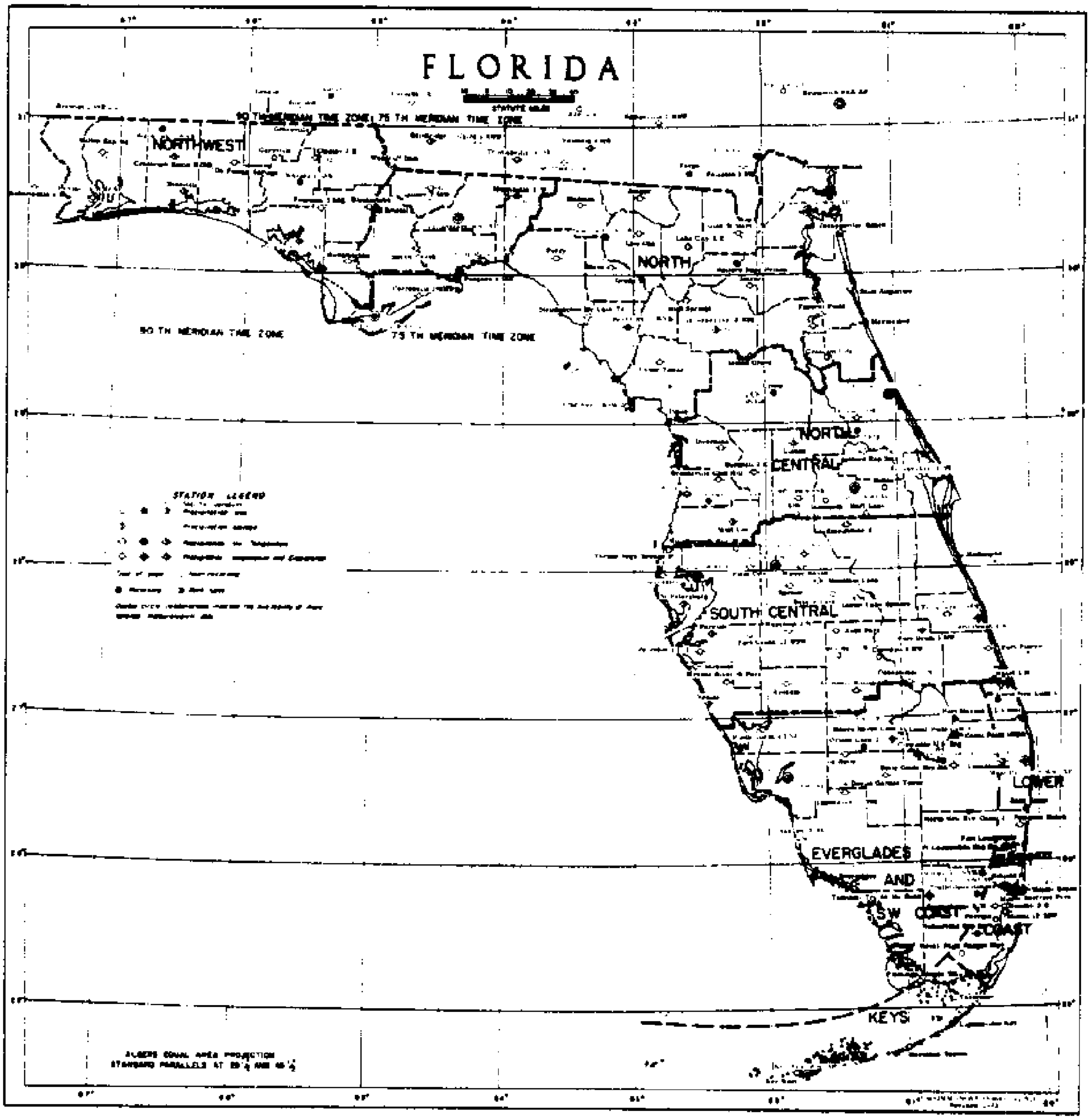


Figure 12.1 - Location Map of Weather and Meteorological Stations in Florida.

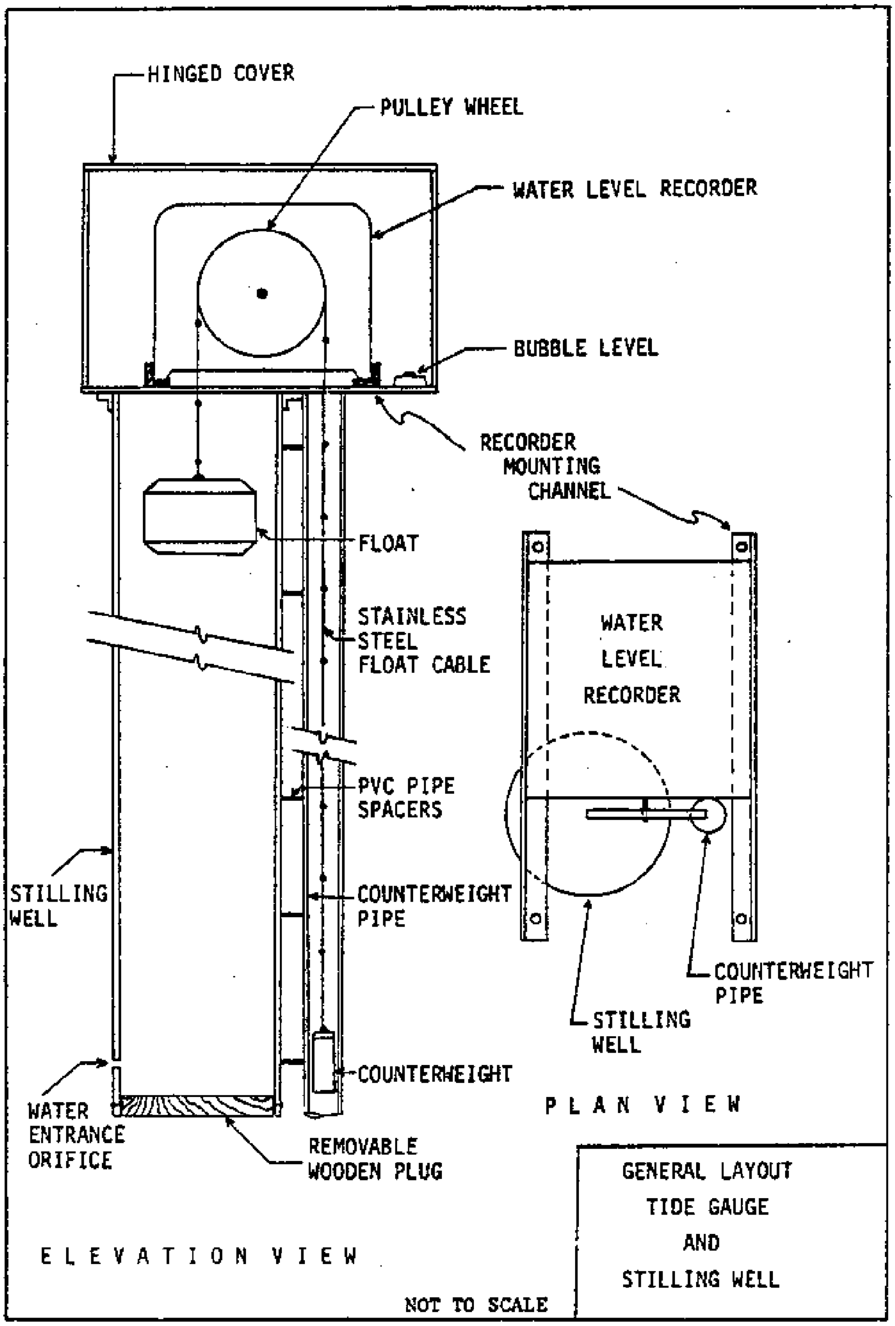


Figure 12.2 - Typical Stilling Well and Water Level Recorder Box for the Tide Measurements.

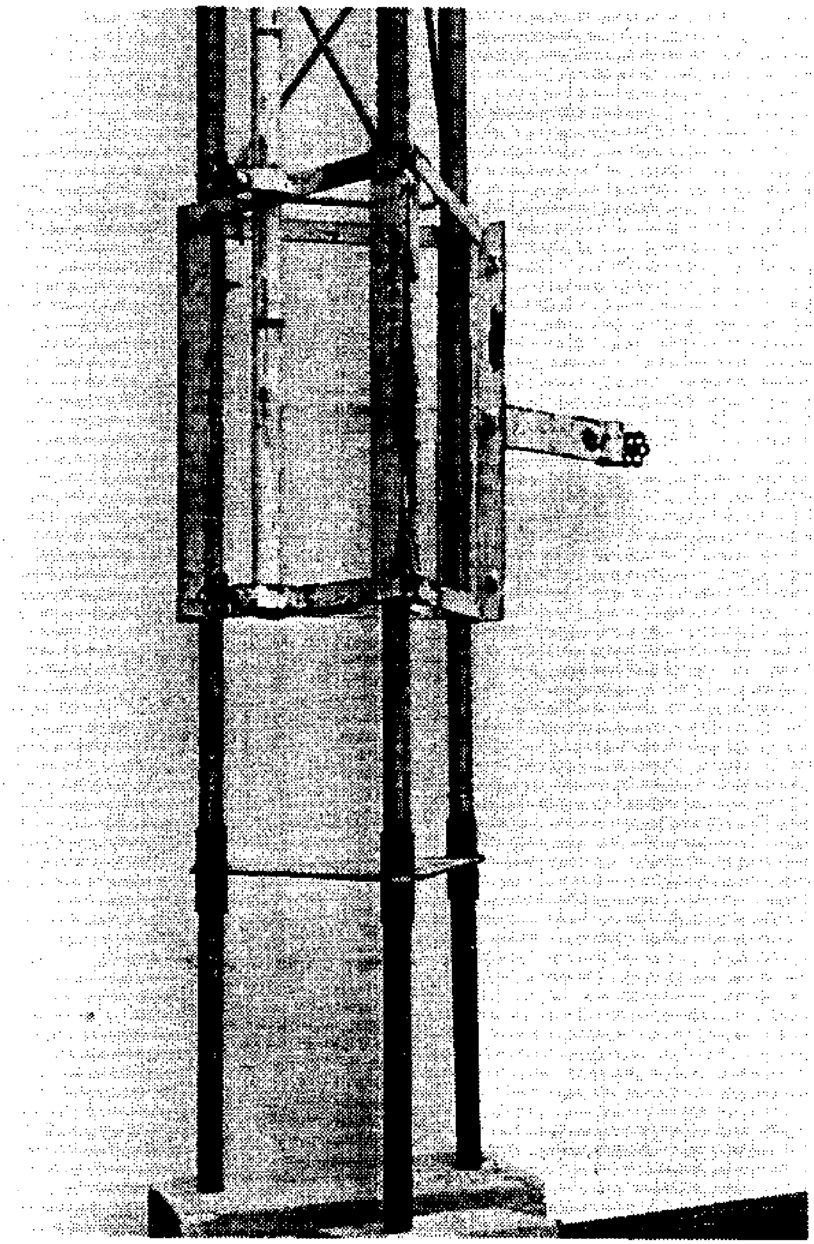


Figure 12.3a - Lower Part of Velocity Tower Showing Carriage With Probe Holder and Adjustable Legs, Designed and Built by Snyder Oceanography Services.

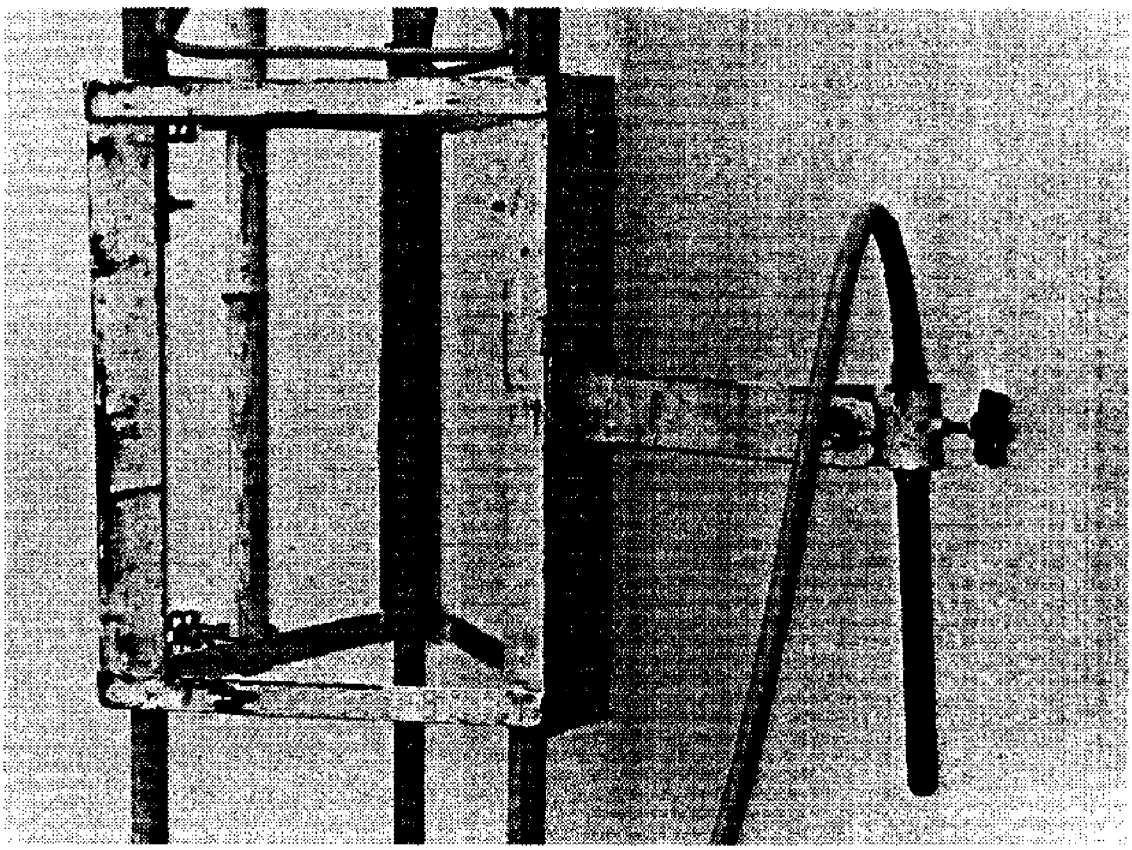


Figure 12.3b - Detail of Probe Holder Carriage on Velocity Meter Tower, With Probe Installed.

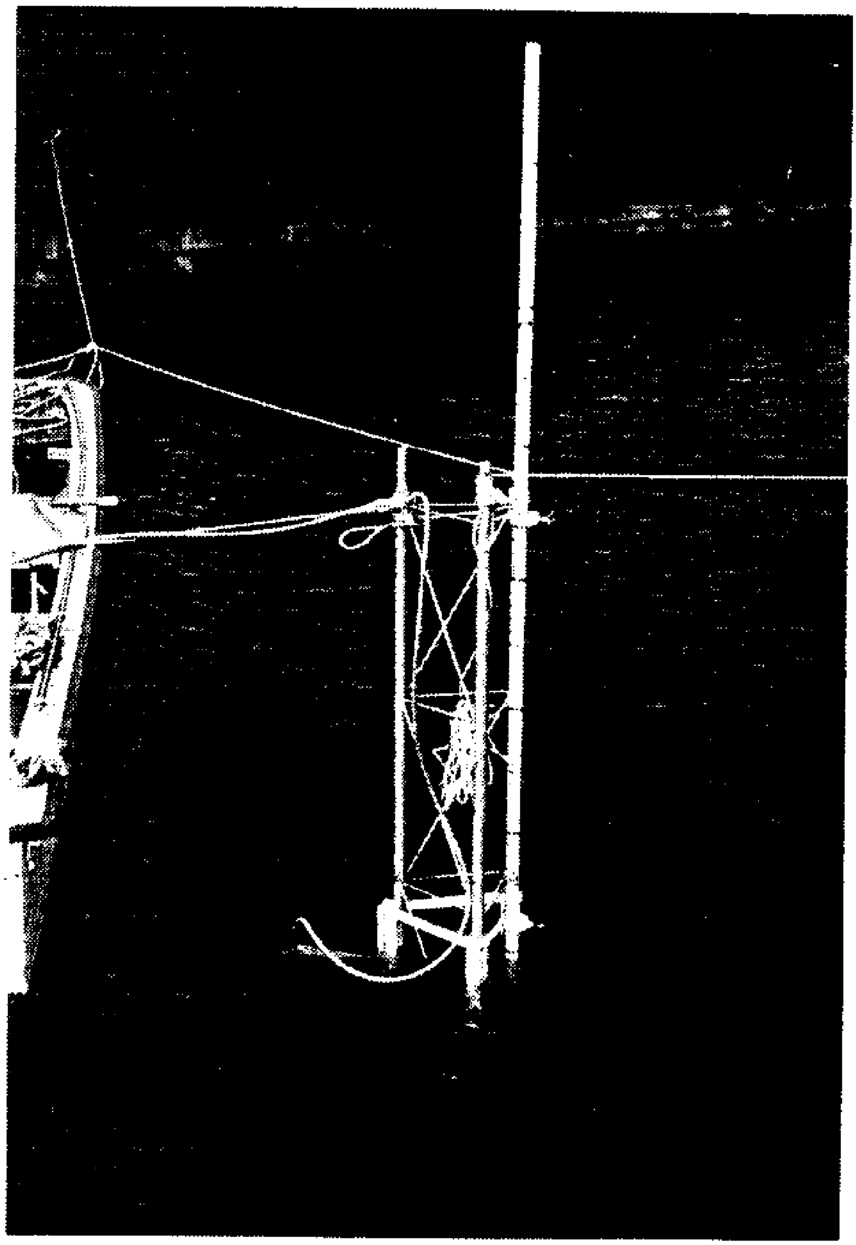


Figure 12.4 - Velocity Meter Tower Set-Up in Canal.

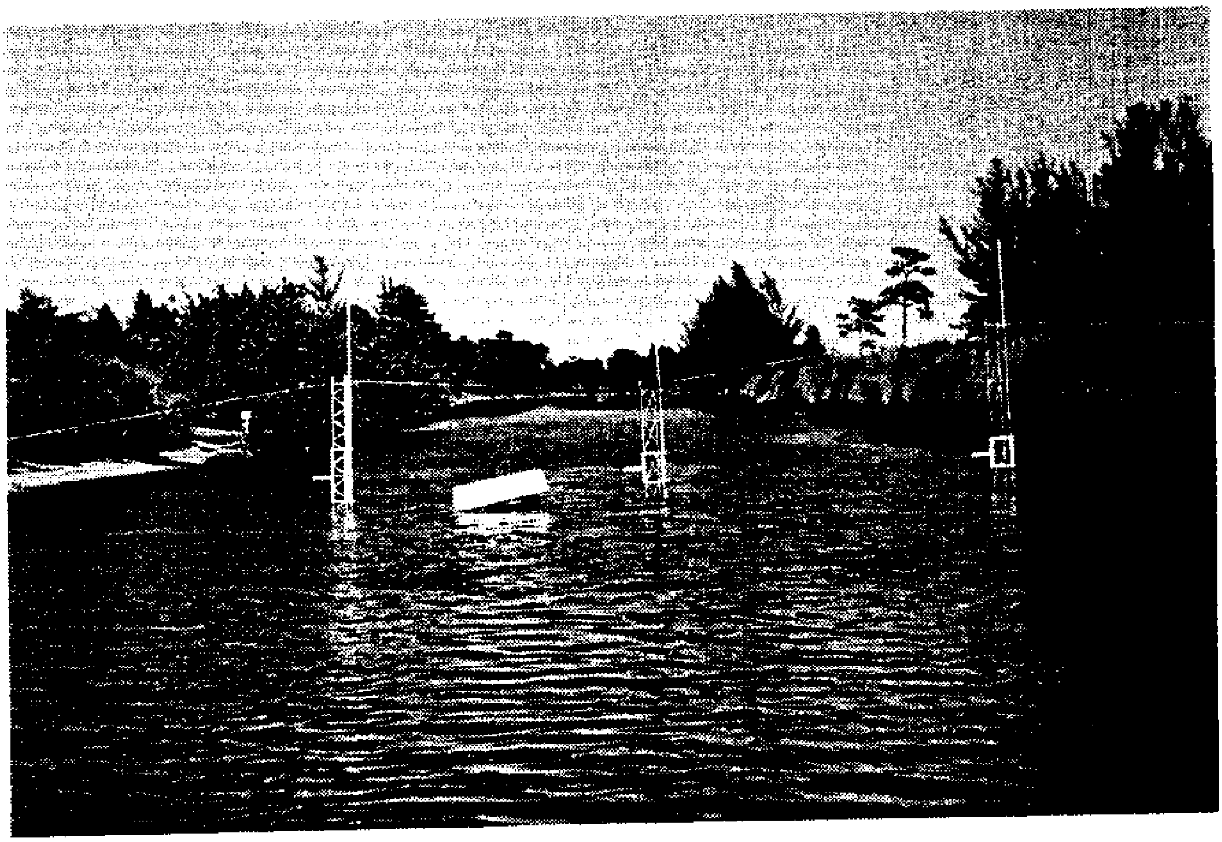


Figure 12.5 - Three Velocity Meter Towers Installed in the Loxahatchee North Canal.

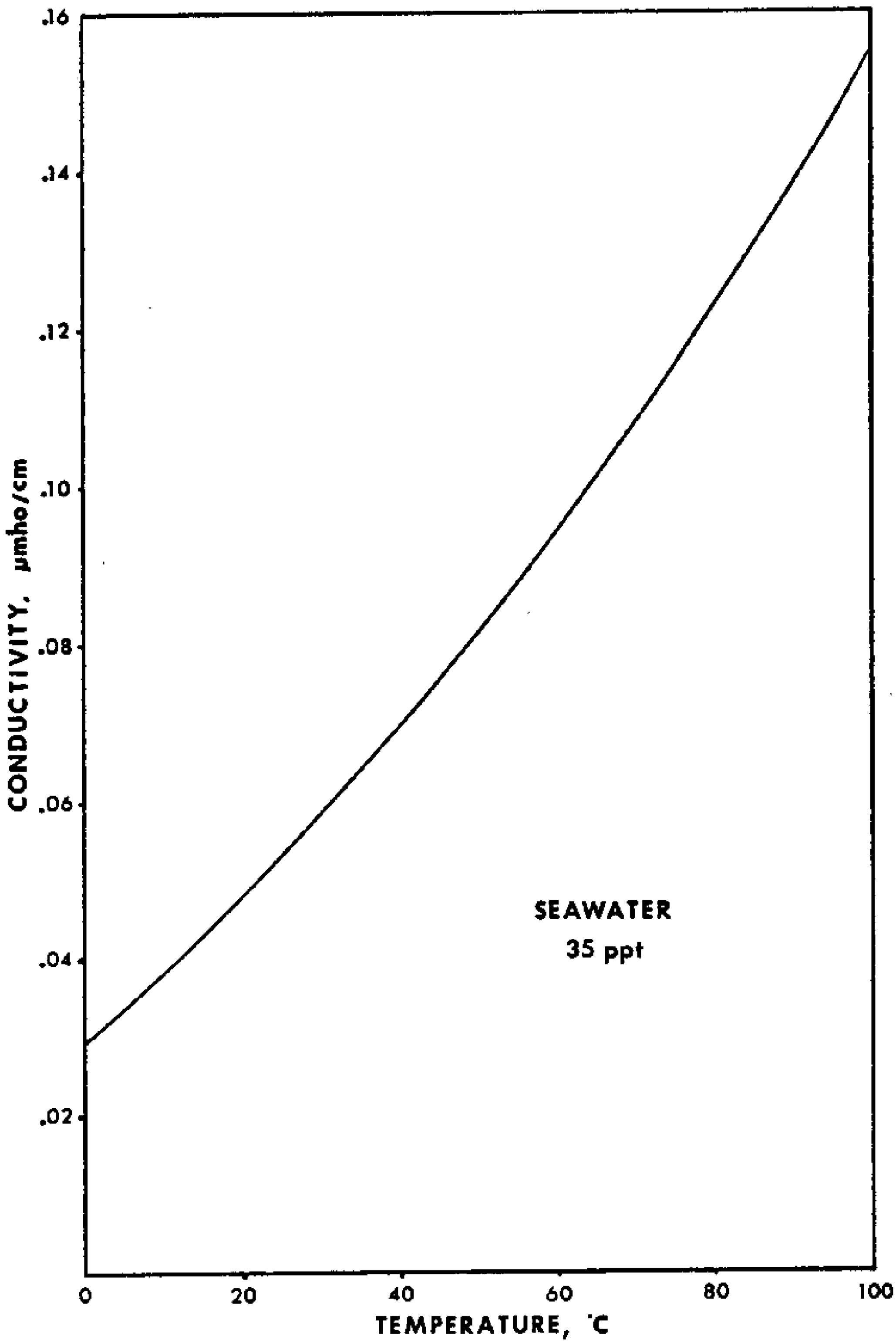


Figure 12.6 - Electrical Conductivity of Seawater as a Function of Temperature (Source: Higgins, L. L., 1962, p. 2.6).

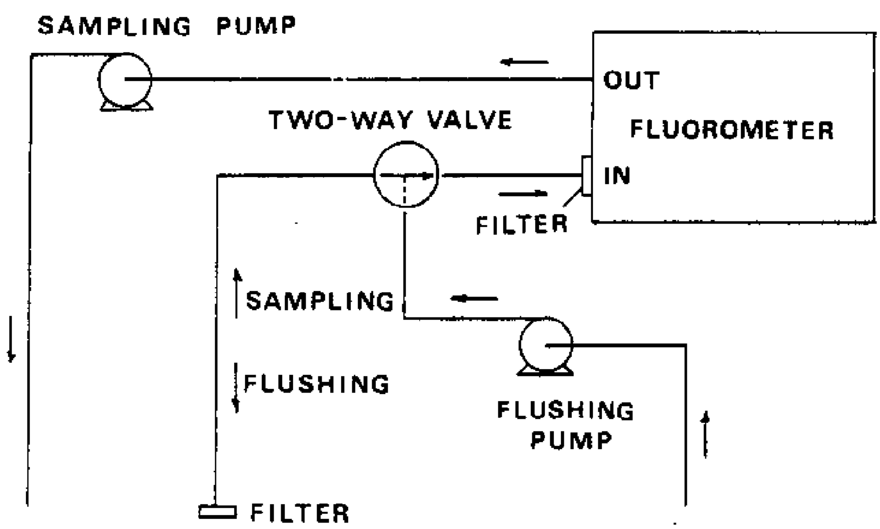


Figure 12.7 - Diagram of Continuous Flow Dye Sampling System.

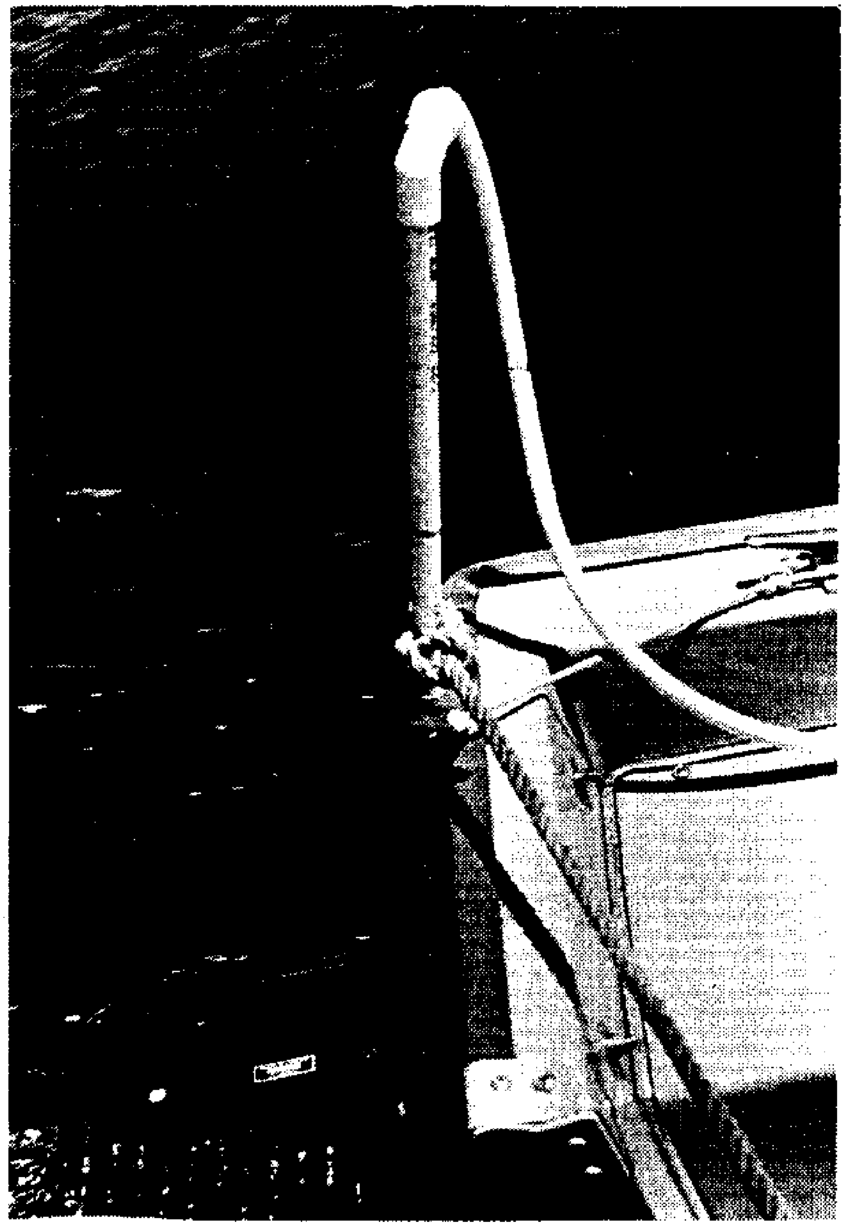


Figure 12.8 - Use of Short Length of PVC Pipe for Sampling Water-Tracing Dye From Moving Boat at 3 ft depth.

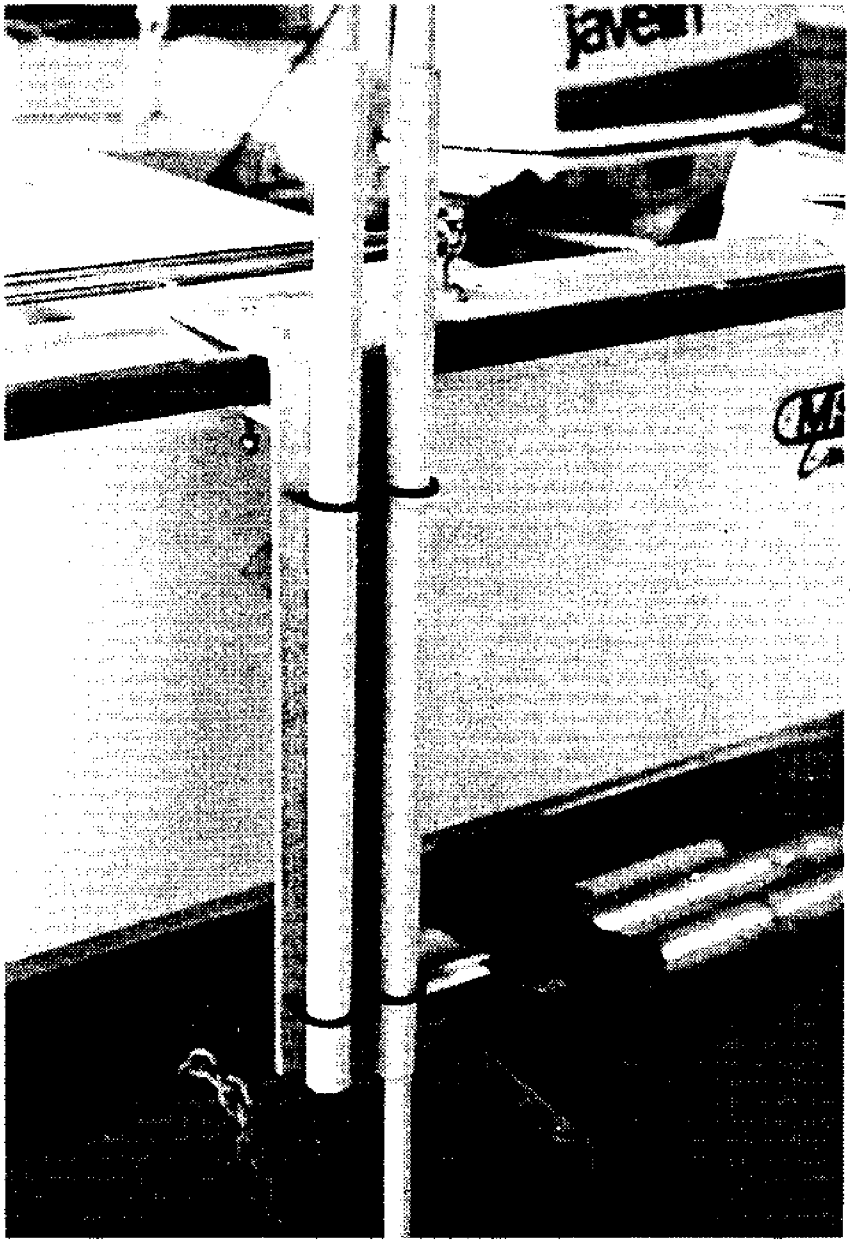


Figure 12.9 - Gunwale Support for 3 and 6 ft Water Sampling Tubes.

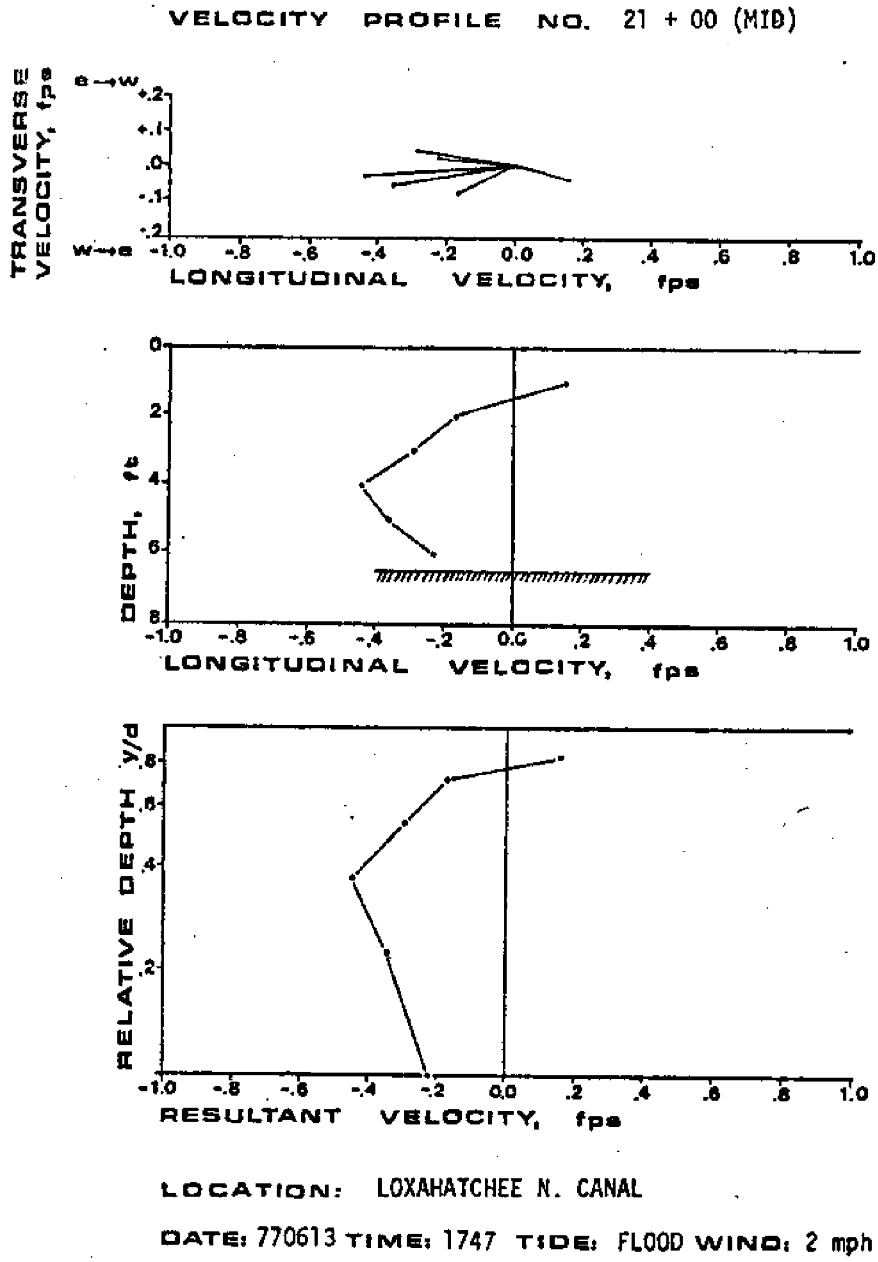


Figure 12.10 - Example of Plotted Velocity Components From Measurements by Electromagnetic Current Meter.

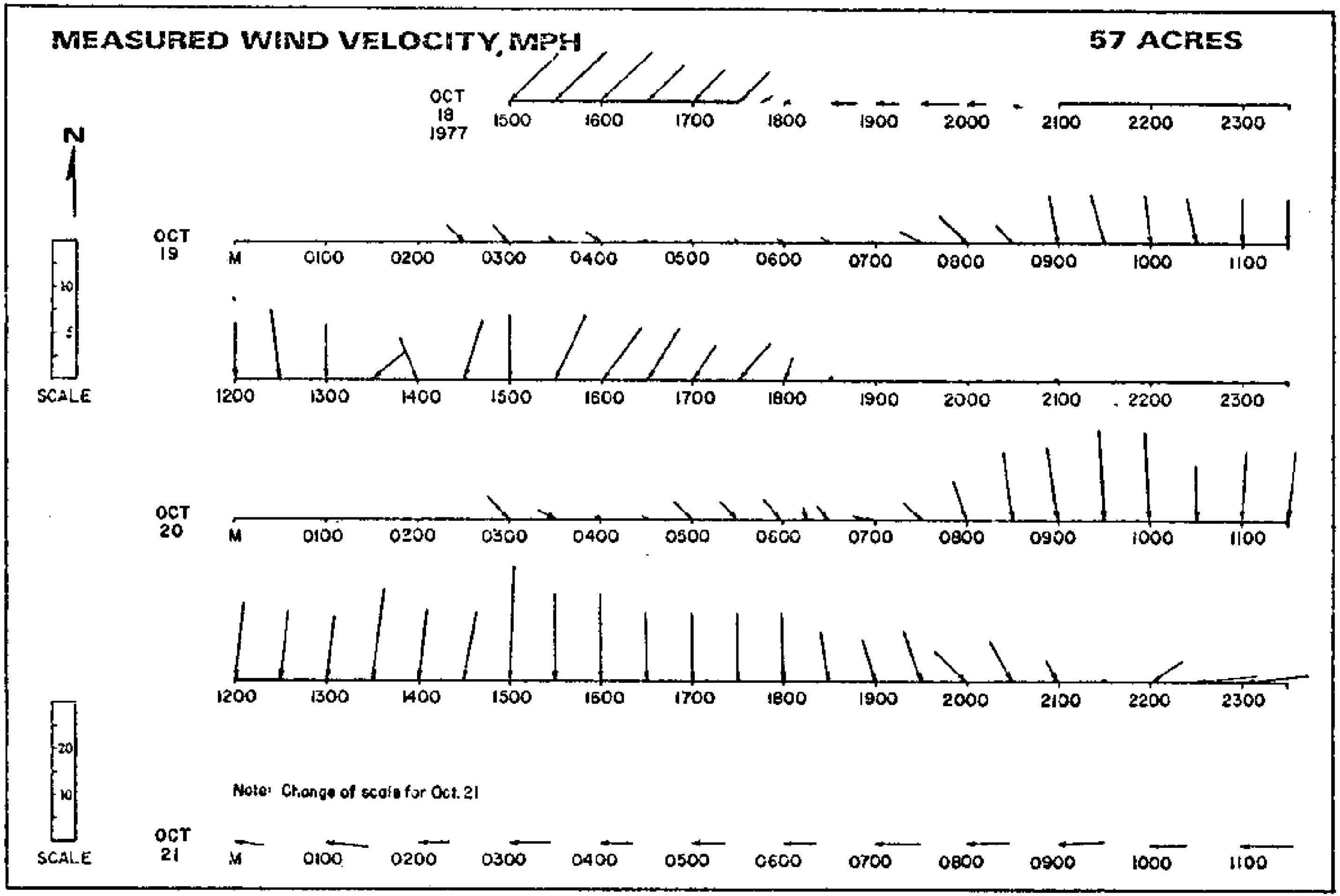


Figure 12.11 - Example of Plotted Wind Velocities From Measurements With Wind Velocity Meter.

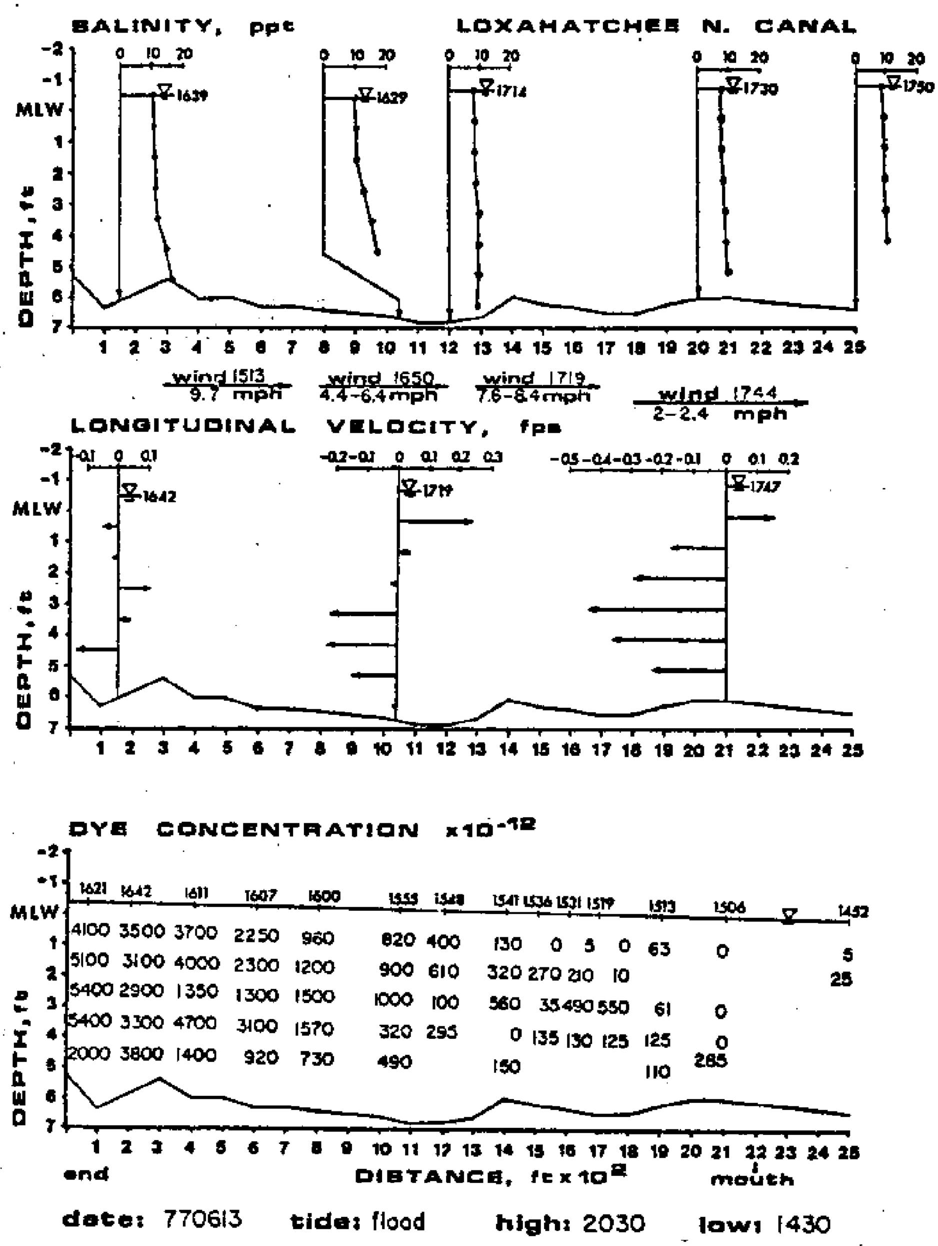


Figure 12.12 - Example of Plotted Salinity, Velocity, and Dye Concentration Profiles.

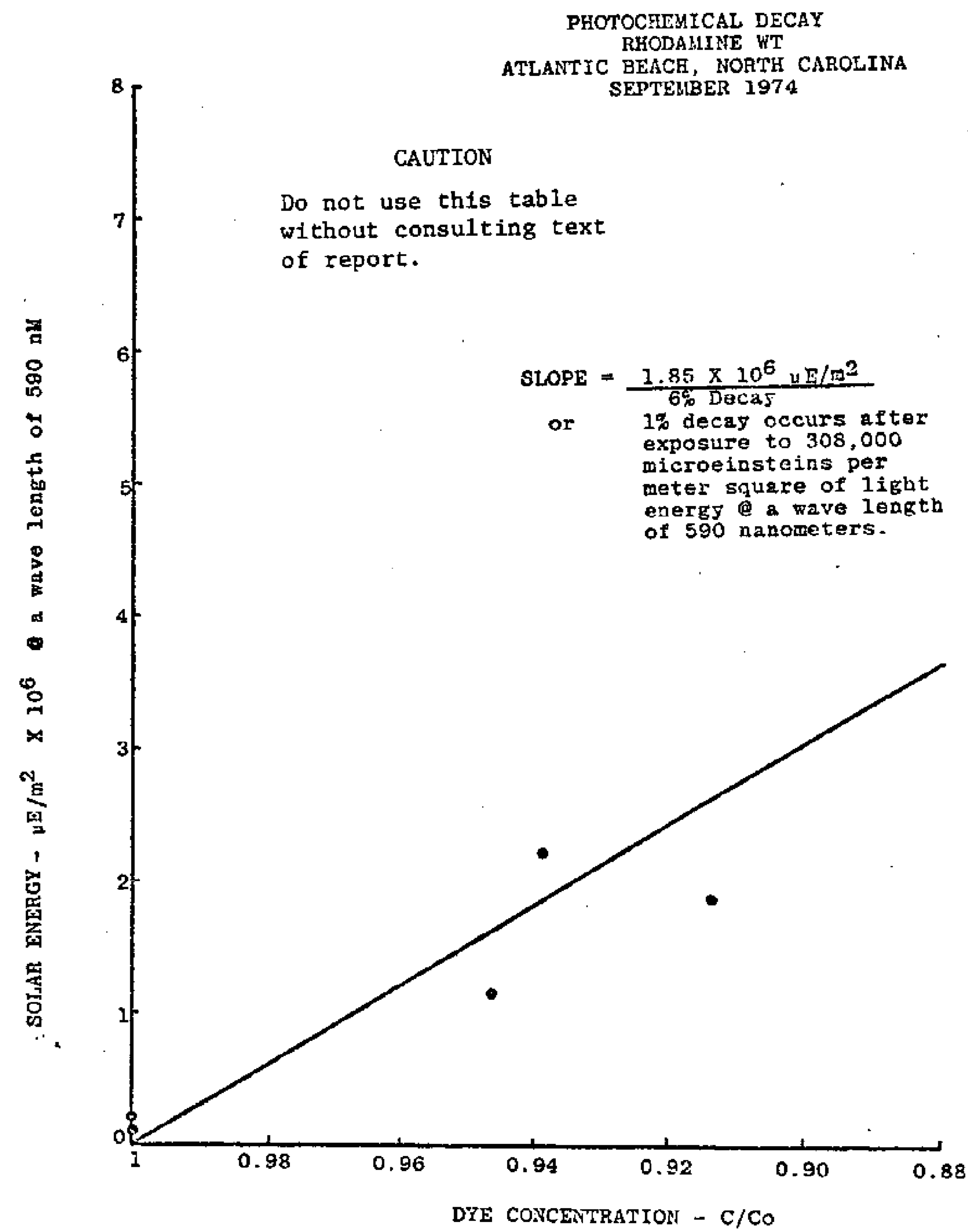


Figure 12.13 - Photochemical Decay Rhodamine WT Atlantic Beach, North Carolina September 1974 (Source: EPA, May 1975, p. 217).

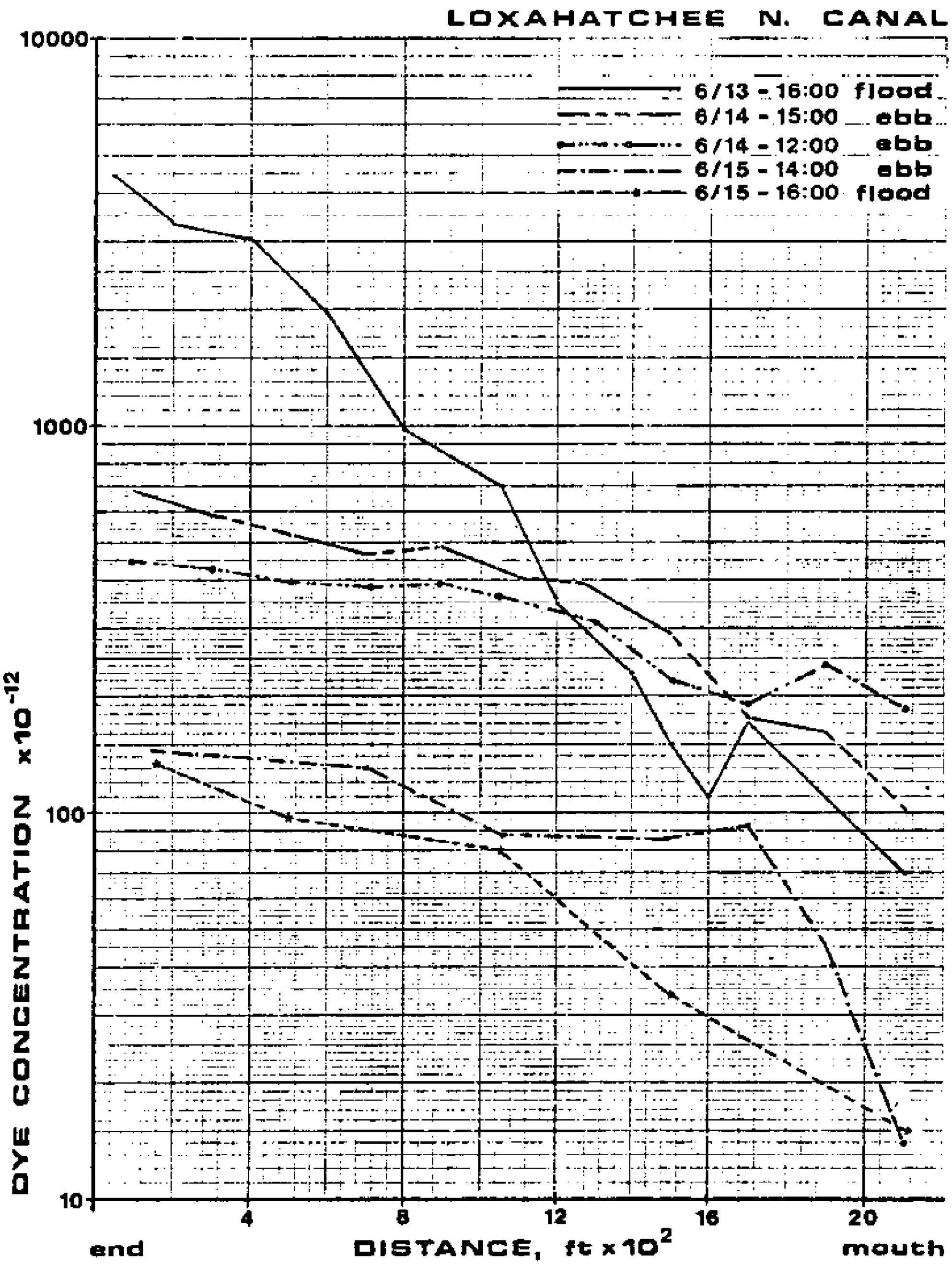


Figure 12.14 - Example of Vertically-Averaged Dye Concentration Profiles.

Table 12.1 - 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 12.2 - 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
	$\tau_{critical}$	

Table 12.3 - List of Field Equipment and Instrumentation, Which Can be Used for Each of the Measured Variables in Table 12.2.

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	Propellor	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 12.4 - 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 industrial activities Particulate matter from industrial stacks, dust, and radioactive particles Material washed from surface of earth, e.g.: Organic matter such as leaves, grass, and other vegetation in all stages of biodegradation Bacteria associated with surface debris (including intestinal organisms) Clay, silt, and other mineral particles Insecticide and herbicide residues
Domestic use (exclusive of industrial)	runoff outfall seepage flow	Undecomposed organic matter, such as garbage, grease, etc. Partially degraded organic matter such as raw wastes from human bodies Combination of above two after biodegradation to various degrees of sewage treatment Bacteria (including pathogens), viruses, worm eggs Grit from soil washings, eggshells, ground bone, etc. Miscellaneous organic solids e.g., paper, rags, plastics, and synthetic materials Detergents
Industrial use	outfall seepage flow	Biodegradable organic matter having a wide range of oxygen demand Inorganic solids, mineral residues Chemical residues ranging from simple acids and alkalies to those of highly complex molecular structure Metal ions

Table 12.4 - continued.

Contributing Factor	Components	Principal Quality Input to Surface Waters
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

Note: Not all sources of pollution in canals are man-made. Some are natural, including the communicating waterbody.

Source: McGauhey, 1968, p. 86.

Table 12.5 - Harmful Effects of Pollutants On Canal Waters and Environment.

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

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

Table 12.6 - 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 12.7 - 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 12.8 - 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 12.9 - 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 12.10 - 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.

CHAPTER 13

DESIGN AND EVALUATION OF TRIAL CANAL NETWORK

In the first twelve 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. Chapter 14 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.

13.1 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 which 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 11, the canal designer will develop a set of quantified design constraints and design criteria (see Table 10.1) which are used in the design of the trial canal network. These steps are summarized in Figure 13.1.

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 other features. The characteristics of these elements can be generalized and quantified to an extent, and therefore such elements can be assembled in various ways to create a canal system. Once assembled, the elements can then be sized to tune the system so that it meets the objectives established for it. Table 13.1 is a list of major design elements and some of their characteristics. Those characteristics which can be quantified are used later on as design constraints (Section 13.2) and design criteria (Section 13.3).

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 should not be changed unless the overall value of the system will be increased and the additional cost of that increase is acceptable to the developer. The term "cost" is used here in its general sense, including the often nebulous environmental cost, and includes the long-term expense to the community as well as the short-term cost involved in the construction and the cost of marketing the product. In general, it is more expensive (i.e., it requires additional energy) to maintain a system which cannot maintain itself in relationship to the other natural systems with which it interacts.

13.2 Quantifying Design Constraints

Design constraints, as described in Chapter 11, Figure 10.1, and Table 10.1, are limitations on the design which 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. Thus, in this case a design constraint would be to allow no depth greater than 15 ft. 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 13.2.

Flushing time may be defined as the time required for an initial, abnormal pollutant distribution to be reduced to some small percentage of the initial value at all points in the canal. There is no standard at present for flushing time, since 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 reasonably accurate for single, straight dead-end canal reaches. The model, then, can be used to predict *comparative* flushing times and to show whether a given change in a design criterion produces an improvement or an adverse effect.

The orientation of the axis of the tidal entrance and the "main channel" connected to it, or any other reach in the network, may be limited to a small sector around the prevailing wind direction by a design constraint. Unfortunately, the most frequent wind direction shifts seasonally. The orientation of entrances and/or channels will therefore be governed by the season and the location in the network where flushing is expected to be the worst. The reaches in the network which are expected to have the most difficulty with flushing should be oriented so that the mean wind direction is upstream or downstream. This may be somewhat difficult to do with restrictive topography or limited surface area to work with.

The bank slope of a reach should be designed, according to the grain sizes of bed and bank materials, to minimize the tendency for accretion or erosion. 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 (see Section 13.4.6).

The vegetative 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 which 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 which could overpopulate the area and become a nusiance.

The shorelines of the canals can be vegetated completely except for a relatively small bulkheaded or riprapped portion. To express this artificial portion as a design constraint, it is more convenient to limit these structures on a per-lot basis (e.g., "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 will usually be via 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.

13.3 Quantifying Design Criteria

Design criteria, as described in Chapter 11, Figure 10.1, and Table 10.1, are statements specifying the value (or range) of a variable which 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 13.3 lists some examples of design criteria.

Beginning with overall system design criteria, a design storm is specified which meets local and state criteria, if any apply. If any of the property lies in a flood plain, it will be advisable to design for the 25-yr flood plain. Outside the flood plain, stormwater retention could, for example, be designed for the 100-yr storm, and drainage without retention for the 25-yr storm, depending on local regulations.

The location and orientation of one or more 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. 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 are used to design 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 13.2, a concept originally conceived 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 will have to be changed as the design evolves.

13.4 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 which are required for operating the mass-transport canal model. Two general classes of problems can be identified at the outset, the first being the case where the site has no existing canal network, and the second where there is an existing network. 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 14.

13.4.1 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 be on hand. 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.

13.4.2 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 (see Section 10.3.2.2). 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 (see references Section 12.3.3.2).

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 which 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 that area which produces a definable freshwater inflow to the site or the waters bordering the site to an extent which would affect either the drainage or the functioning of the canal network.

The synthesis of hydrographs is described in detail in the literature [Christensen, et al, 1974; Hjelmfelt and Cassidy, 1975; Viesman et al, 1972; SCS, 1972]. An example is given in Appendix I. If the SCS method is used, the basic steps involved are:

1. Define drainage basin and point or points at which surface or ground water flow affect the site. The drainage basin may be wholly or partially outside the boundaries of the site. The hydrograph may have to be developed for portions of the site itself, in conjunction with the drainage plan, as well as for the offsite area.
2. In conjunction with step 1, develop a drainage plan for the site, at least showing the direction of surface water movement.
3. From "soil survey interpretations" for soils at the site, select "runoff curve numbers", evaluate effect of antecedent moisture conditions and adjust runoff curve numbers as necessary.
4. Evaluate time of concentration from hydraulic length of drainage basin and velocity of surface flow from average slope of drainage basin.
5. Evaluate design rainfall depth, P, which is design rainfall intensity multiplied by storm duration and find direct (synthesized) runoff, Q (inches), from Figure 10.1 in SCS Handbook [1972].

6. Determine hydrograph family number, duration of rainfall excess T_o, time to' peak T_p, T_o/T_p ratio, revised time to peak, unit flow q_p, and peak flow Qq_p.
7. Determine discharge q_c versus time.

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, and using the rational method, based on the equation,

$$Q = CIA \tag{13.1}$$

where

Q = peak discharge, cfs

C = a runoff coefficient (dimensionless)

I = rainfall intensity, inches/hour

A = drainage area, acres.

This model takes some time to set up on the 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 [1975] would be suitable.

13.4.3 Friction Effect From Measured Velocity Profile

The friction effect is quantified by means of the variable, k, which is Nikuradse's equivalent sand roughness (dimension L). This variable is defined as the size of the roughness elements along the wall of a closed conduit which is completely covered by a surface of these uniform roughness elements. When the flow through such a conduit is steady and uniform, the velocity profile will be logarithmic and the value of k at any cross-section is the diameter of the roughness element. Extending this concept to unsteady (tidal) flow in a trapezoidal channel, the value of k turns out to be on the order of approximately 1 to 20 ft or more. This empirical coefficient can be measured if at least the lower part of the velocity profile, near the bed, is logarithmic. This has been shown, by measurements, to be a reasonable assumption for flow unaffected by wind and salinity gradients in Floridian canals, and is justified because at the point where the velocity profile differs from logarithmic, the shear stress, τ_0, which is proportional to the velocity gradient, $\frac{\partial u}{\partial y}$, is unaffected by the type of profile above this point. It is only affected by the total depth and the profile close to the bed of the canal.

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

$$\frac{u}{u^*} = 2.5 \ln\left(\frac{29.7y}{k}\right) \tag{13.2}$$

where

y = vertical distance from bed, (L)

u = velocity at distance y, (L/T)

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

k = Nikuradse's equivalent sand roughness, (L).

The bed shear velocity, u^*, can be developed from a measured logarithmic profile by selecting two points, P_1 and P_2, on the profile and finding the corresponding velocities, u_1 and u_2. If u_1 is the velocity at $y_1 = P_1 d$ and u_2 is the velocity at $y_2 = P_2 d$, then

$$\frac{u_1}{u^*} = 2.5 \ln \left(\frac{29.7 P_1 d}{k}\right) \tag{13.3a}$$

$$\frac{u_2}{u^*} = 2.5 \ln\left(\frac{29.7 P_2 d}{k}\right) \tag{13.3b}$$

Adding Equations (13.3a) and (13.3b) gives

$$u^* = \frac{u_1 - u_2}{2.5 \ln \frac{P_1}{P_2}} \tag{13.4}$$

Nikuradse's equivalent sand roughness, k, may now be found from either Equation (13.3a) or (13.3b),

$$k = \frac{29.7 P_1 d}{e^{\frac{u_1}{2.5u^*}}} \tag{13.5a}$$

or,

$$k = \frac{29.7 P_2 d}{e^{\frac{u_2}{2.5u^*}}} \tag{13.5b}$$

or by combining Equations (13.3a) and (13.3b),

$$k = \frac{29.7P_1 d}{\frac{\left(\frac{u_1}{u_1 - u_2}\right)}{\frac{P_1}{P_2}}} \tag{13.6}$$

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

$$\tau_o = \rho(u^*)^2 \tag{13.7}$$

For example, for the logarithmic velocity profile, VW5, taken at point VW in the 57 Acres system (Figure 13.3) the spatial mean velocity, $\bar{u}$, is 0.153 fps, bed shear velocity, u*, is 0.071 fps, bed shear stress, τ_o, is 0.0098 psf, and roughness, k, is 13.4 ft.

13.4.4 Probablity of Erosion or Deposition

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

$$\tau_{cr \cdot h} = A_h(\gamma_s - \gamma)d_e \tag{13.8}$$

where

A_h = Shield's entrainment coefficient (dimensionless)

= function of wall Reynolds number

γ_s = unit weight of bed material, $[FL^{-3}]$

γ = unit weight of water, $[FL^{-2}]$

= 64.18 lb/ft^3 for saltwater

d_e = effective grain size of bed material, [L] (usually expressed in mm)

$$= \left(\int_{y=0}^{1} \frac{dy}{d}\right)^{-1} \tag{13.9}$$

y = the fraction by weight of the sediment that is finer than d.

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

The effective grain size is obtained from a sediment sample by first thoroughly drying the sample, weighing the total sample, weighing successively smaller samples passed by finer sieves and plotting the fractions in percent finer than each quantity on the ordinate versus grain size on a logarithmic scale on the abcissa (Figure 13.4). The grain size corresponding to the sieve diameter which passes 35 percent of the material, called $d_{35\%}$, is approximately equal to the effective grain size of the sample. A more accurate determination of the effective grain size may be obtained by fitting three or more straight lines to the grain size distribution curve and calculating d_e by the formula,

$$\frac{1}{d_e} = \Sigma_{n=1}^{p} \frac{y_n - y_{n-1}}{\ln \frac{d_n}{d_{n-1}}} \left[\frac{1}{d_{n-1}} - \frac{1}{d_n}\right] \qquad (13.10)$$

where

p = number of approximating straight lines

d = diameter of grain (mm)

y = fraction finer than value, by weight.

As an example, the grain-size distribution curve for station B has been given in Figure 13.4. The effective grain size, $d_{35\%}$, is 0.155 mm (5.09×10^{-4} ft), specific weight difference ($\gamma_s - \gamma$) is 92.0 lb/ft^3 (see Table 13.4), and therefore from Equation (13.8),

$$\tau_{cr \cdot h} = 0.056 \times 92.0 \times 5.09 \times 10^{-4}$$
$$= 0.0025 \text{ psf}$$

Erosion will theoretically begin on the bed of a channel when the time-mean bed shear stress $\bar{\tau}_o$ exceeds $\bar{\tau}_{cr \cdot h}$. Measurements of vertical velocity profiles in an existing channel, or predictions of mean velocity from a tidal prism analysis, permit values for $\bar{\tau}_o$ (see Section 13.4.3) to be calculated.

The critical shear stress on a bank can be related to the critical horizontal shear stress by several methods. For channels which are hydraulically rough the effect of hydrodynamic lift on the sediments can be neglected. The criterion for ignoring the lift is met if the ratio of the equivalent sand roughness, k, to the effective grain size, $d_{35\%}$, is greater than or equal to 100. To evaluate this ratio from typical velocity measurements in a Floridian canal, all of the *logarithmic* vertical velocity profiles (five out of a total of eleven) taken at station VW at the 57 Acres site may be considered. From these data (Table 13.5) the five values of the ratio, r,

$$r = k/d_{35\%} \tag{13.11}$$

are 69 600, 26 300, 26 300, 8 800, 11 600 for a $d_{35\%}$ value of 0.155 mm (5.09×10^{-4} ft). These results justify the use of the USBR (Lane's) formula,

$$\bar{\tau}_{cr \cdot b} = \cos\theta\sqrt{1 - \frac{\tan^2\theta}{\tan^2\phi}}\;\bar{\tau}_{cr \cdot b} \tag{13.12}$$

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

θ = bank slope angle, measured from horizontal [degrees]

ϕ = angle of repose of material, [degrees].

For example, for an angle, ϕ, of 30° and using the measured bank slope, s, of 1:7, angle, θ, is $\cot^{-1}s$ or 8.13°,

$$\begin{aligned}\bar{\tau}_{cr \cdot b} &= \cos 8.13°\sqrt{1 - \frac{\tan^2 8.13°}{\tan^2 30°}}\,(.00262)\\ &= 0.929(.00262)\\ &= 0.0024 \text{ psf}\end{aligned}$$

The critical bank shear stress can be predicted in a similar manner using relationships developed by Christensen [1971]. The time mean critical bank shear stress, $\bar{\tau}_{cr \cdot b}$, is given by,

$$\bar{\tau}_{cr \cdot b} = [F(s,\phi,r)]\bar{\tau}_{cr \cdot h} \tag{13.13}$$

where

F = function of ( )

s = inverse bank slope (dimensionless)

ϕ = angle of repose of bank material, [degrees]

r = $k/d_{35\%}$ (dimensionless)

The relationship for a ϕ of 30° and r less than 100 is shown in Figure 13.5. Since all of the measured bed shear stresses $\bar{\tau}_0$ at station VW (Table 13.5) are greater than both the bank and horizontal critical shear stresses (except for station VW4, where $\bar{\tau}_0$ is 0.0015 psf), this is indicative that erosion was occurring at the time of the measurements. Additional plots of the results of Equation (13.13) for other values of r are available in Christensen [1971].

The velocities and shear stresses on vegetated banks will be lower than on nonvegetated banks, since the bed shear velocity, u^*, is approximately proportional to the mean velocity (the increase in roughness, k, having a minor effect on the proportionality constant for a logarithmic velocity profile, Equation (13.2)) and the bed shear stress is proportional to $(u^*)^2$. Thus, a vegetated bank will have less tendency to erode both from a hydrodynamic viewpoint and due to the additional stabilization provided by the roots of the vegetation. The bank slope design should therefore be based on providing a stable base for revegetation, and a greater critical shear stress than the bed shear stress that would result from the design storm. The additional predicted lateral inflow resulting from a storm can be simulated by the mass-transport model (CANNET3D) to find predicted vertical velocity profiles, which can then be used to find the required bank slope.

13.4.5 Stable Cross-section Design

A flow-through channel, such as a channel connecting two tidal entrances, should be designed not only to minimize erosion and deposition, but also to accommodate a certain discharge. In general, it is well known that a natural alluvial channel will adapt its depth, width and slope, no matter what the original dimensions are, to a specific set of values for a particular discharge and water/sediment complex. The question of stable section design in a dead-end residential canal is moot, because the stable cross-section for such low flows is on the order of a 5 or 10 ft width and a fraction of a foot depth. Such dimensions are frequently seen in natural tidal creeks in Florida. For this reason, an artificial dead-end canal will always have a tendency toward deposition, and maintenance dredging will be periodically required.

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

Christensen [1976] has developed a design technique which provides a value for the mean depth, d_o, and the bottom width, b, for a mild-slope trapezoidal channel with cohesionless alluvial or marine deposits, given discharge, Q, bed slope, S_b, bank slope, s, effective grain size, d_e, equivalent sand roughness, k, and the probability-dependent critical shear stress of the bed material on a horizontal bed, $\overline{\tau}_{cr\cdot h}$. Figure 13.6 is entered with a trial value for the quantity,

$$\frac{d_o}{\left[\frac{Q}{M\sqrt{S_b}}\right]^{3/8}} \tag{13.14}$$

where

d_o = approximate mean depth, [L]

Q = mean discharge, $[L^3T^{-1}]$

$M = \frac{1.49}{n} = \frac{8.24\sqrt{g}}{k^{1/6}}$, $[L^{1/3}T^{-1}]$

n = Manning's n

S_b = bed slope (dimensionless)

and at its intersection with the design value for inverse side slope, s, the value of the ratio, b/d_o, is taken from the abcissa. If the resulting value of bottom width, b, is not suitable, the procedure may be iterated. The corresponding value for the time-mean critical bank shear stress is,

$$\overline{\tau}_{cr\cdot b} = \gamma S_b s \tag{13.15}$$

The shape of a stable bank profile designed in this way is shown in Figure 13.7 for various values of angle of repose and for correction factor, r, equal to five. For other values of r, the following equation can be used:

$$\frac{dY}{dX} = \tan\phi\sqrt{1 - Y\sqrt{1 + Y'^2}\left[\frac{L}{L + \cot\phi} + \sqrt{1 + [L^2 - \cot^2\phi]Y'} \frac{\cot\phi}{L + \cot\phi}\right]} \tag{13.16}$$

where

$Y = y/y_h$ (dimensionless)

$X = x/y_h$ (dimensionless)

$y_h = \bar{\tau}_{cr\cdot h}/\gamma S_b$, [L]

$L = 0.556\ [\ln(\frac{10.4}{r} + 1)]^2$, (dimensionless)

ϕ = angle of repose (degrees)

$Y' = dY/dX$

13.5 Trial Canal Design

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 which 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 13.8 shows the arrangement of these network designs, and their dimensions are summarized in Table 13.6.

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 13.9).

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 13.10. The 50 ft bottom width and 8 ft mean depth are typical for Floridian canals, and with a side slope of 3:1 the mid-tide surface width is nearly 100 ft.

13.5.1 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 Chapter 8. Since the decrease in concentration from the no-wind distribution increases 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 tidal entrance downwind of the prevailing wind direction.

The flushing simulations in these canal networks begin with a 0.0 or 1.0 ppm background concentration , depending on the particular simulation to be conducted, equal to the receiving waterbody concentration at all points in the network. The pollutant inflow is set at a discharge of 0.04 ft^3/ft-hr at a concentration of 100 ppm at each longitudinal 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 comparisions to wind conditions.

13.5.2 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 2.1.3.1). The dimensions of the channels and the designation of reaches and junctions are shown in Figure 13.11. 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 13.12 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 13.13. 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. Figure 13.14 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 13.15. 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 13.14 shows this effect quantitatively in reach number 1.

Figure 13.16 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 useful information will be gained by extending the simulation.

13.5.3 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 13.17.

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 13.18). A direct comparison between this case and no-wind conditions for System A (Figure 13.12), 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 13.19 and 13.14.

The effect of superimposing a wind blowing into the finger canals is summarized in Figure 13.20. 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 13.19, 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 13.21 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.

13.5.4 Comb-structured Network With Bends

The third network, System C (Figure 13.22), 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 .

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

$$\frac{\alpha}{1 - \cos\frac{\alpha}{2}} = \frac{L}{D_o} \tag{13.17}$$

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,

$$r = L/\alpha \tag{13.18}$$

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 along the chord of the finger canals, upstream. Figure 13.23 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 13.15, 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 be 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 improved upon by a two-dimensional analysis.

13.5.5 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

$$L = \sqrt{gd_0}\, t \tag{13.19}$$

where

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

d_0 = mean depth, (L)

t = time associated with phase lag, (T).

In this case the flood tide flows through both entrances simultaneously and the water surface remains approximately horizontal as the network fills (or empties). It is reasonable then to assume that in some area in the middle of the network the incoming water from the two entrances will come together, but that there will be very little effect from one entrance on the other half of the system. The lack of a sustained unidirectional current (in the absence of wind) in the null zone (called a null point in the model), over a substantial part of the tidal cycle would cause the two parts of the system to flush more or less independently. There would also presumably be enough mixing in such an area, due to interaction of the small but oppositely-directed tidal currents and the variable effect of shifting winds, to overcome any tendency toward stagnation.

In the initial tidal prism analysis of the 57 Acres canal network [Walton, et al, 1975, pp. 38-43], which was conducted before the mass-transfer model, CANNET3D, had been developed, such an area was assumed to exist and it was called a "zone of interaction". Two different methods were considered for finding the possible location of this area. The first was based on the assumption that it would be centered at the point which divided the canal network into two equal water areas. This may be a valid assumption if the canal network is symmetric from the null point to either of the two entrances, but for an asymmetric network such as 57 Acres, somewhat questionable. The second method was based on the measured time lag between the tidal peaks at the entrances and at the two dead-ends as described in Section 2.1.3.3. Both of these methods placed the zone of interaction on the main channel within an area on the main loop between the junction with the reach leading to the "lake" and just south of the "West Branch/North" junction

(Figure 2.8). However, there is some doubt that measurements of tidal lag using tide gauges are sufficiently accurate to provide a reliable result by this method.

The mass-transport canal model (CANNET3D) can be used to simulate flushing in a network with two tidal entrances (i.e., a "loop") provided that the tides at the entrances are in phase or nearly in phase and the system does not have a null point within a second-order or higher reach. System D, shown in Figure 13.24, 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 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 the area.

The results of no-wind conditions on this network are shown in Figures 13.25a and 13.25b. 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 13.12). The comparison in Table 13.7 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 13.26a and b. In this case, both surface and bottom concentrations are on the order of half of the values in the no-wind case.

13.5.6 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

$$x' = \frac{2a}{d_o + a} L \tag{13.20}$$

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 out along the bottom layer occurs under these conditions, but flushing is slow. The flushing of the main (in this case, the north/south) canal is 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 outer portions of finger canals, due to increased convection, but little effect on dead-ends.

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 computing with small numbers.

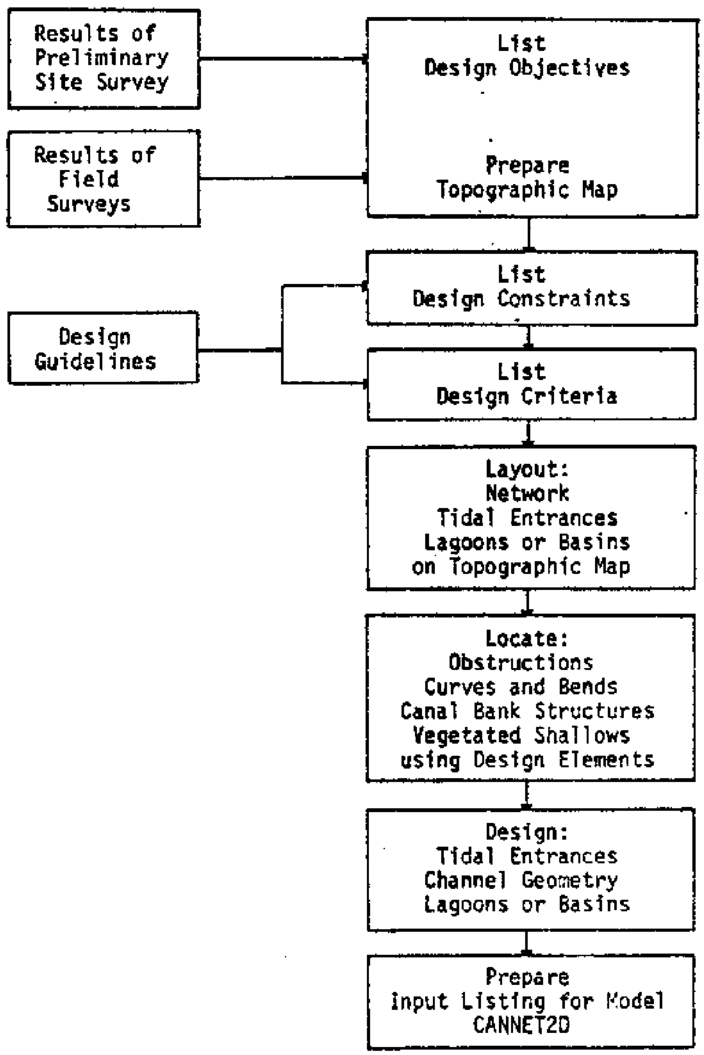


Figure 13.1 - Steps in Formulating a Trial Canal Design.

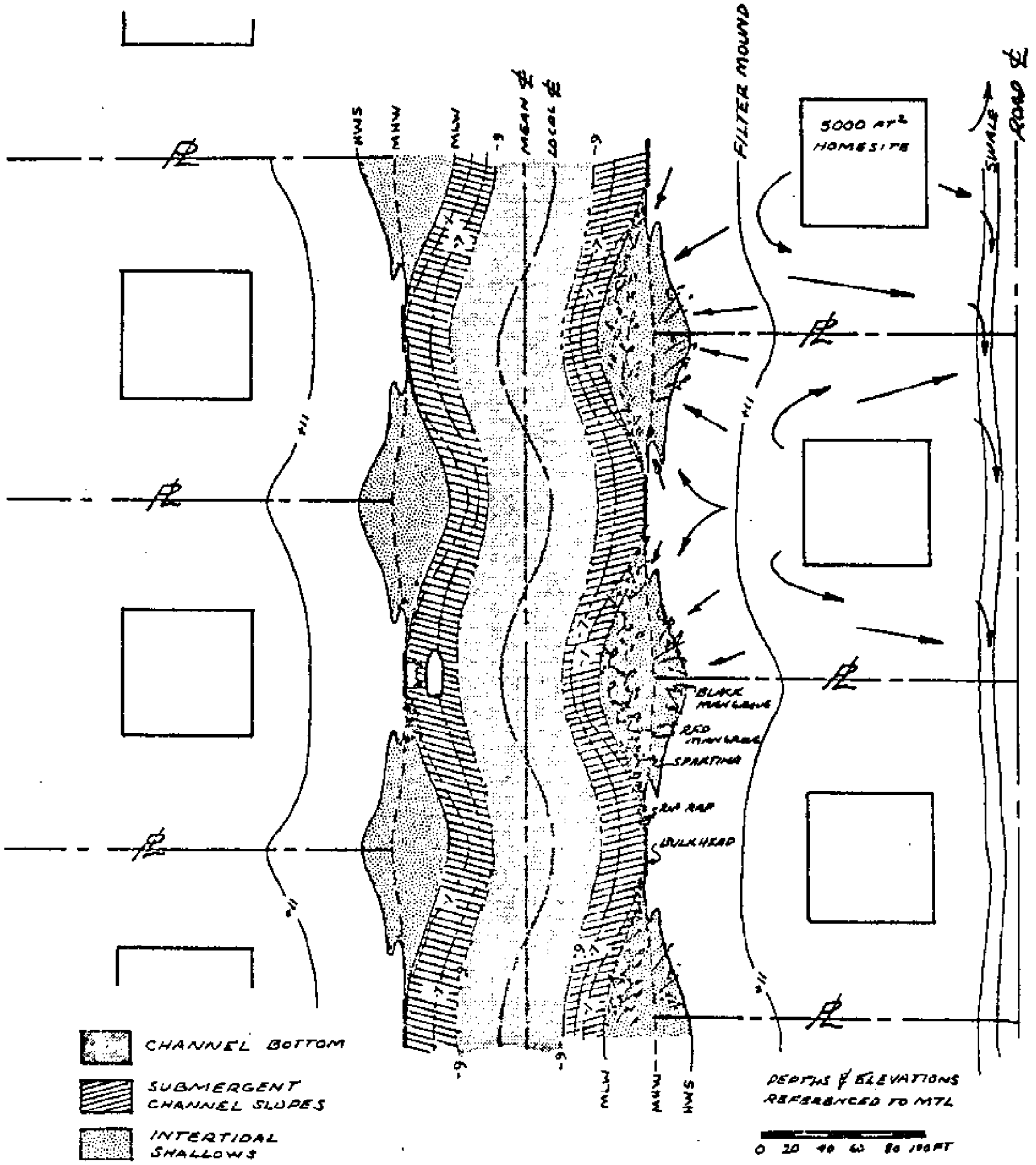


Figure 13.2 - Sinuous Bank Design for 57 Acres Project. (Source: Snyder, R. M., 1976, p. 77).

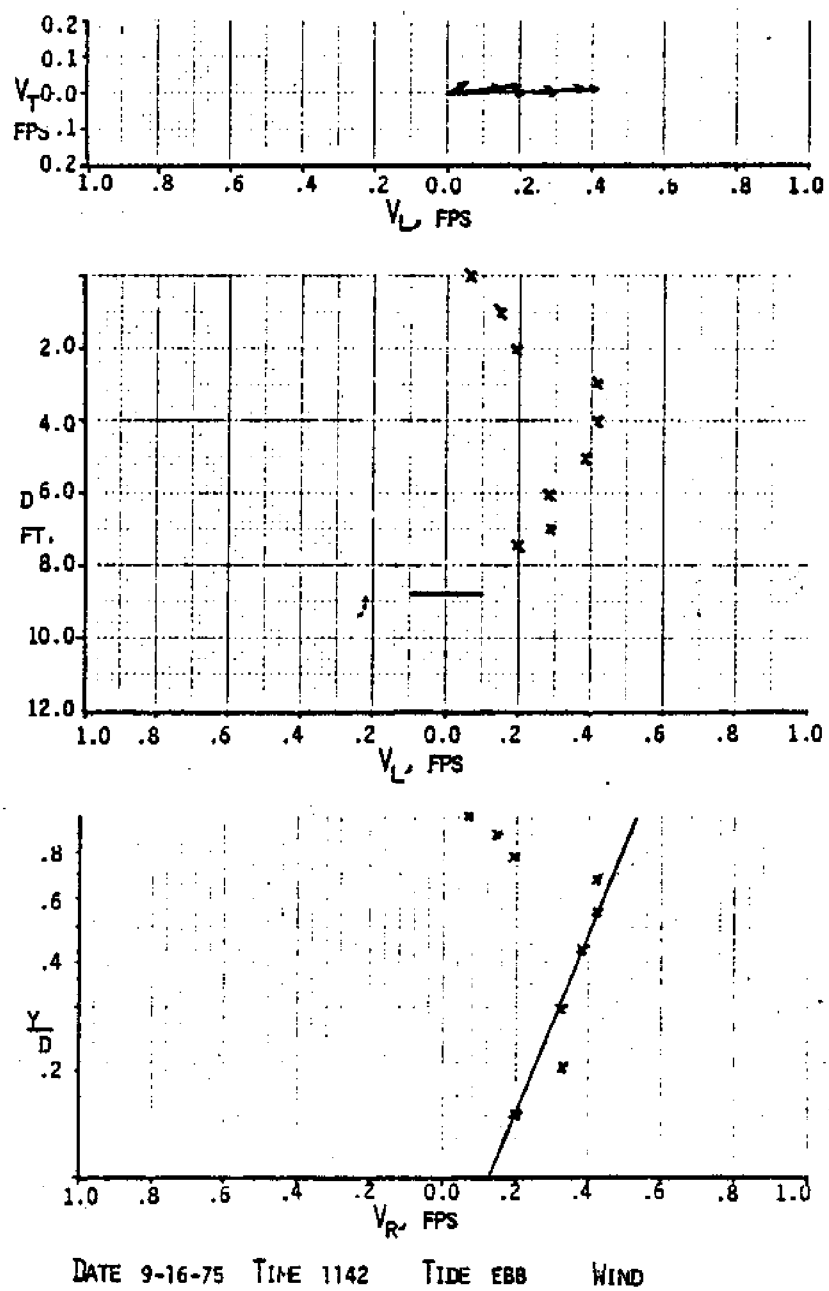


Figure 13.3 - Measured Velocity Profile at Station VW5, 57 Acres Canal System.

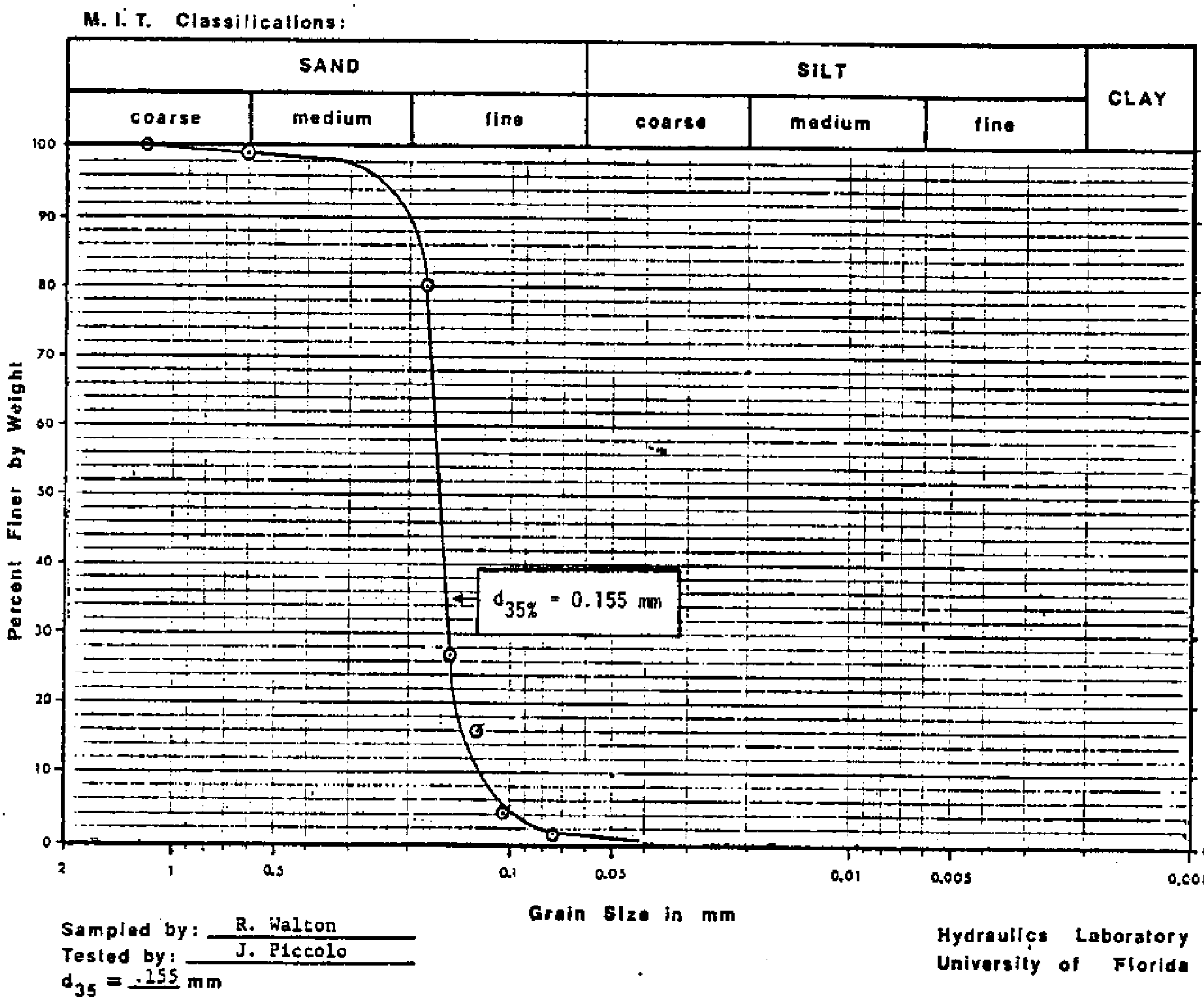


Figure 13.4 - Measured Bed Grain-size Distribution Curve for Sample From Station B, 57 Acres Canal System.

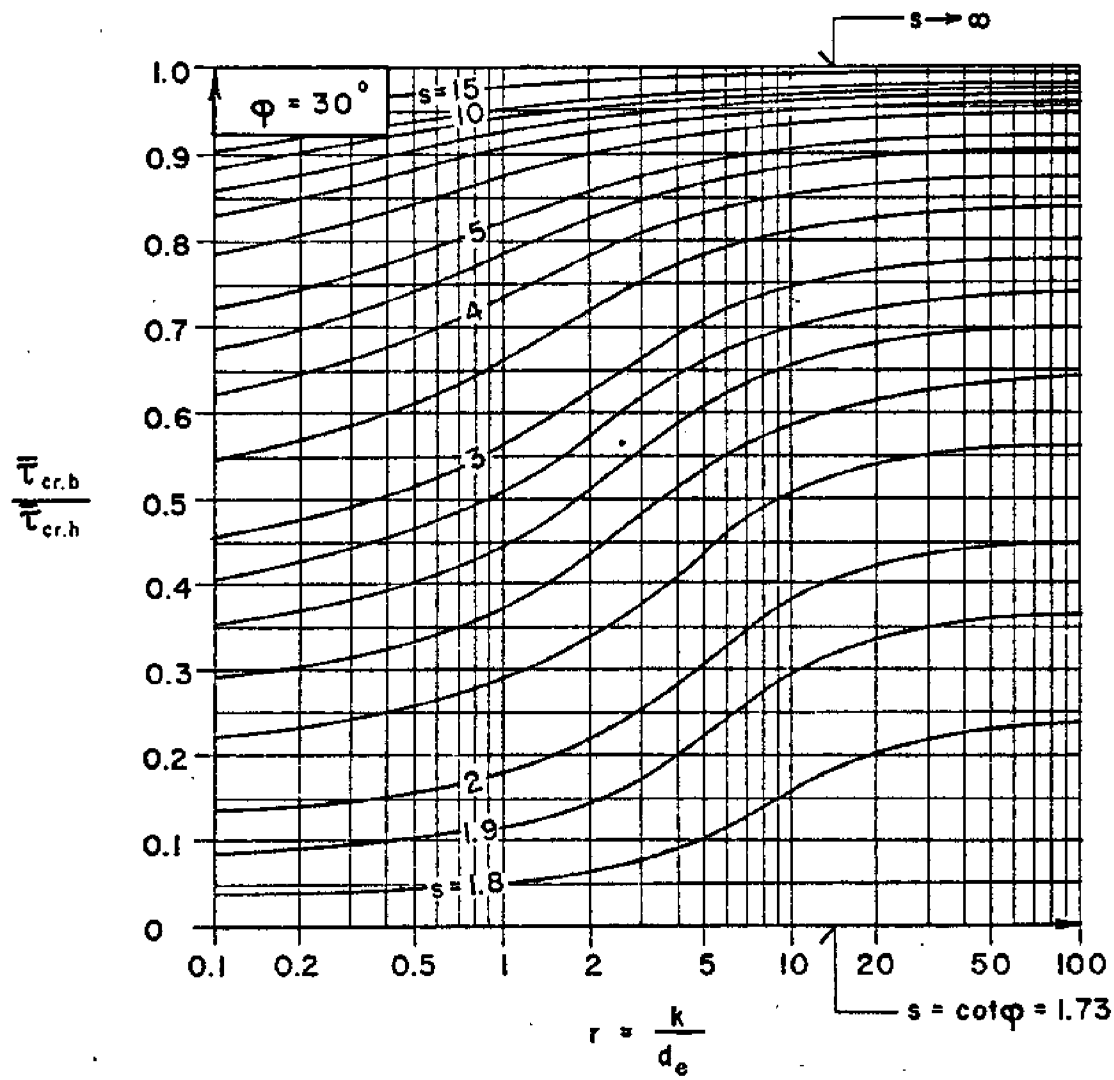


Figure 13.5 - Critical Shear Stress Ratio $\bar{\tau}_{cr \cdot b}/\bar{\tau}_{cr \cdot h}$ as a Function of Correction Factor, r, and Inverse Bank Slope, s.

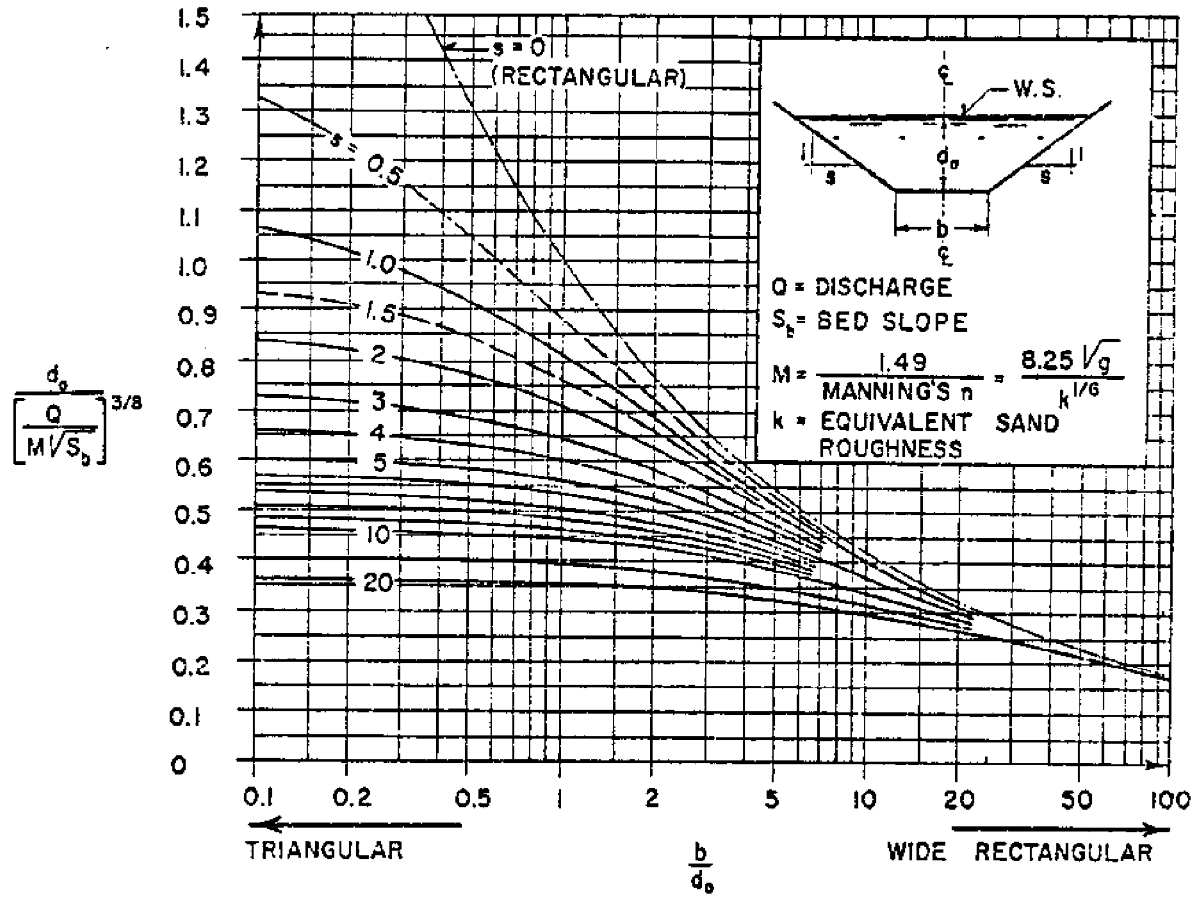


Figure 13.6 - Design Chart for Trapezoidal Channels.

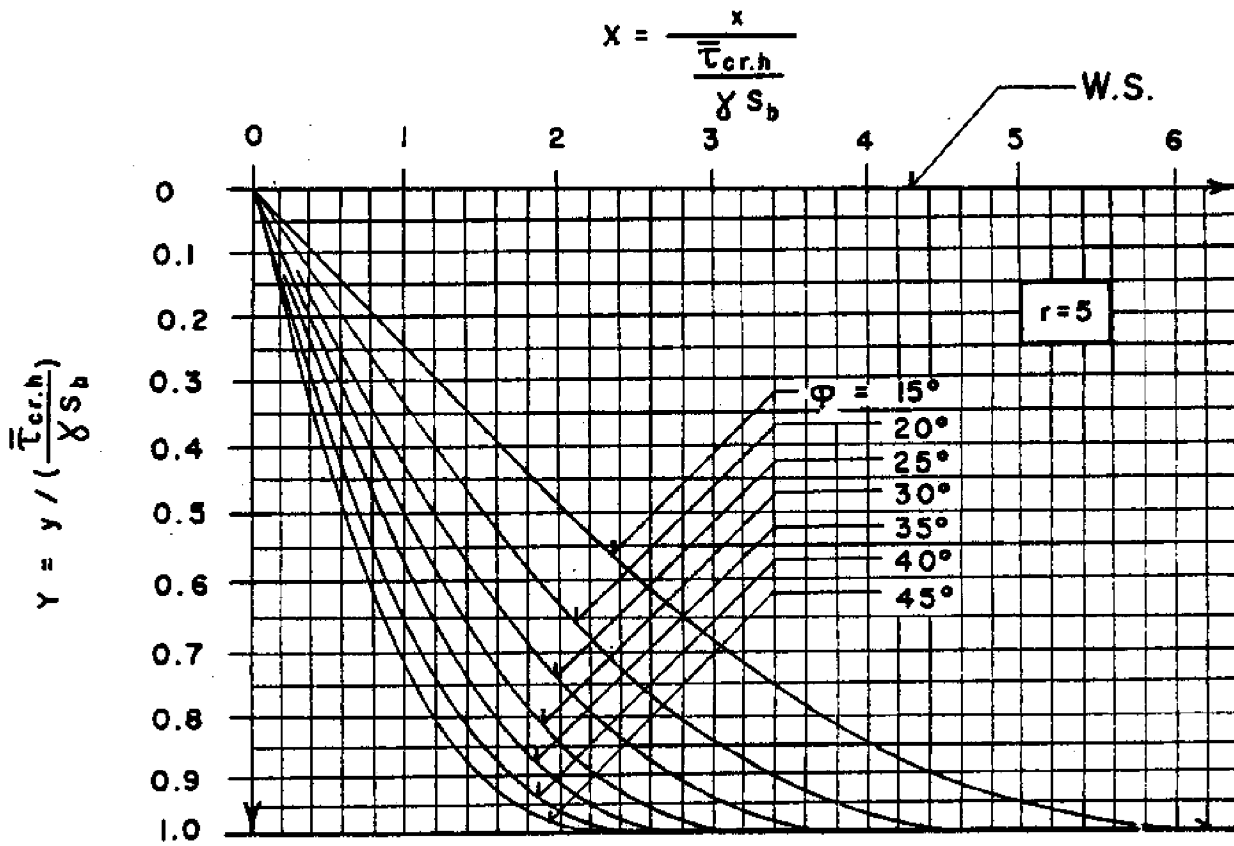


Figure 13.7 - Stable Bank Profile, Mild Slope, Cohesionless Material, r = 5.

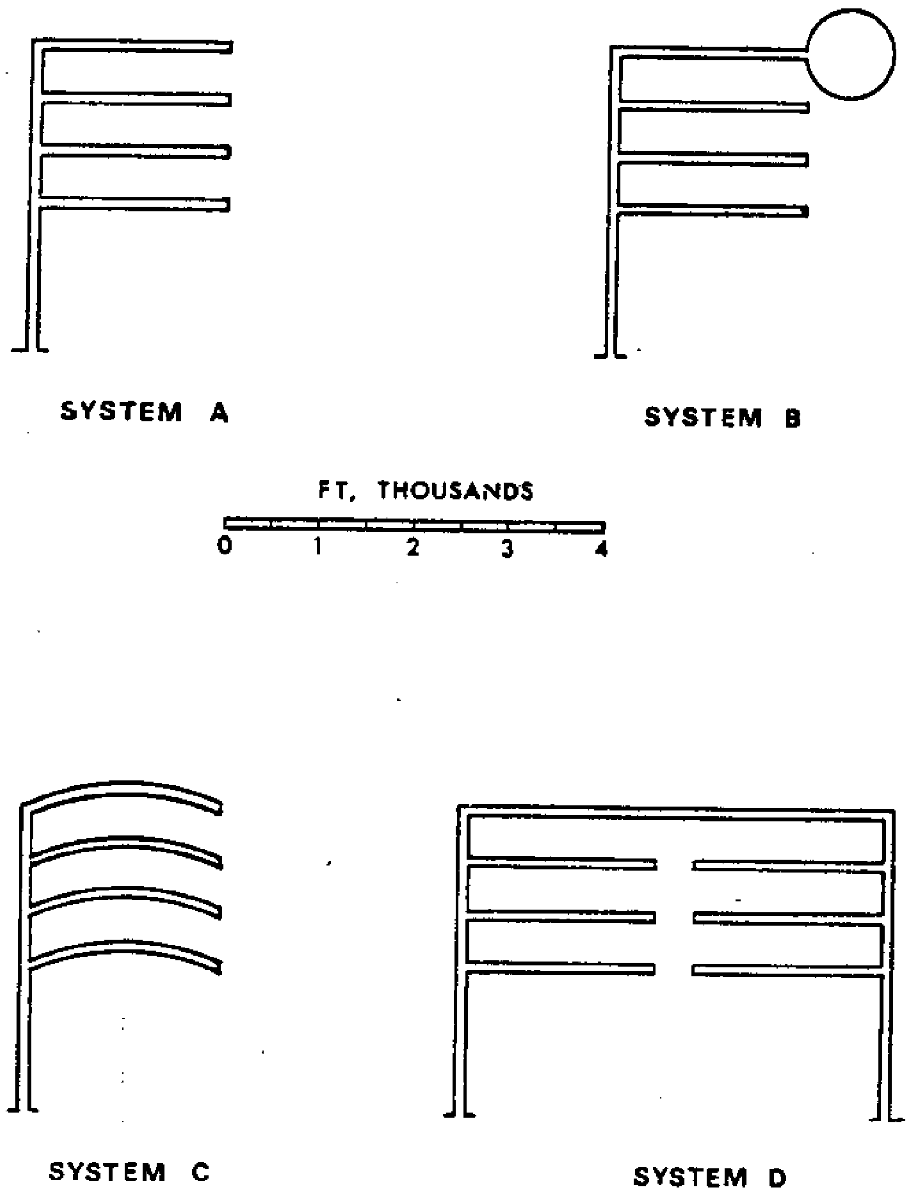


Figure 13.8 - Four Simple Network Design Elements Tested With the Mass-Transfer Model CANNET3D.

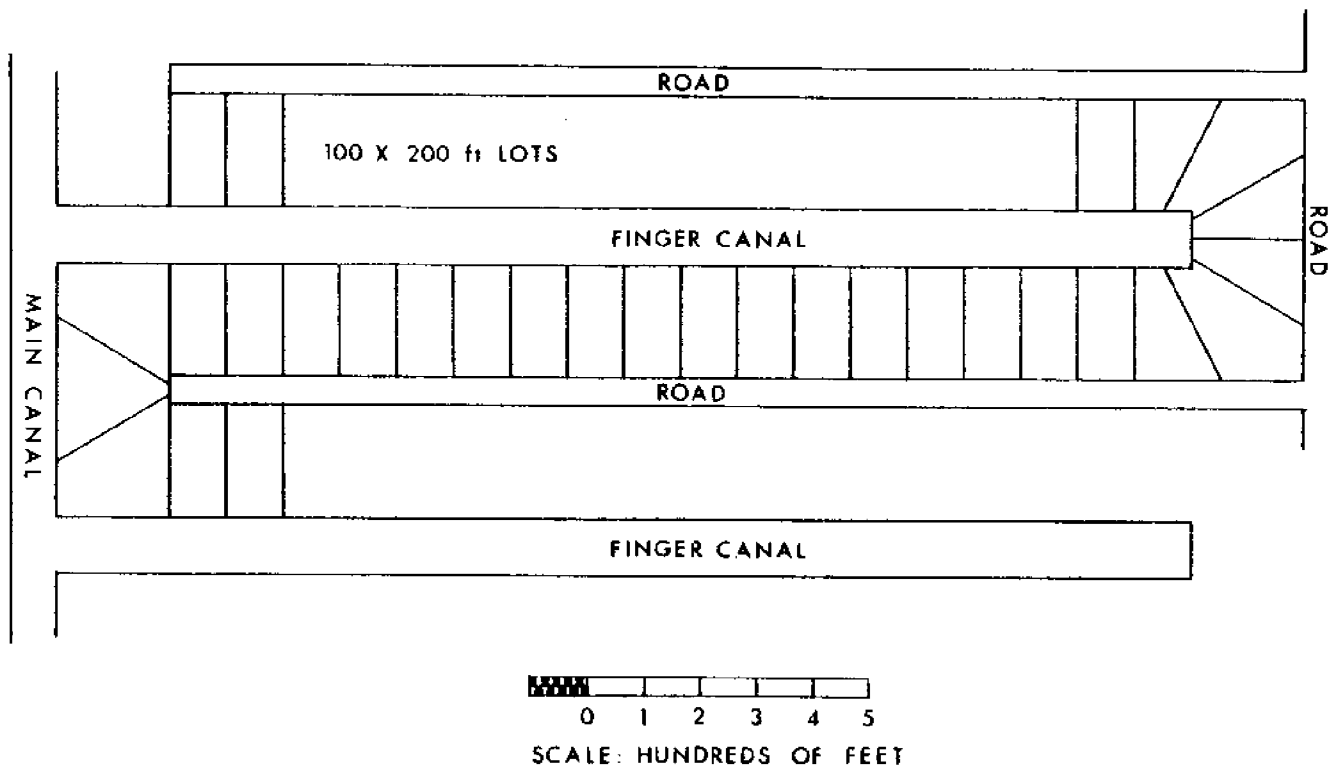


Figure 13.9 - Test Canal and Lot Dimensions.

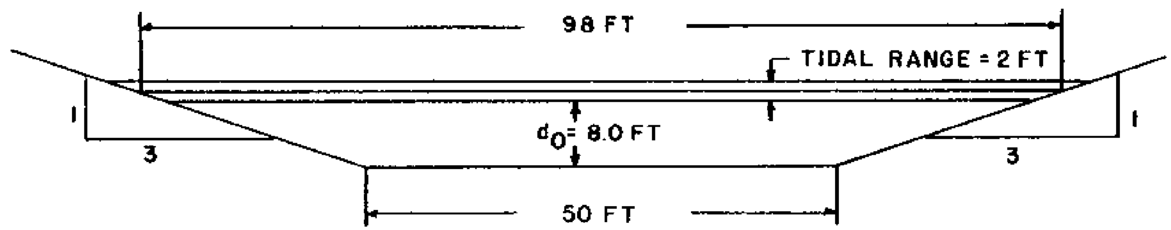


Figure 13.10 - Test Canal Cross-Section.

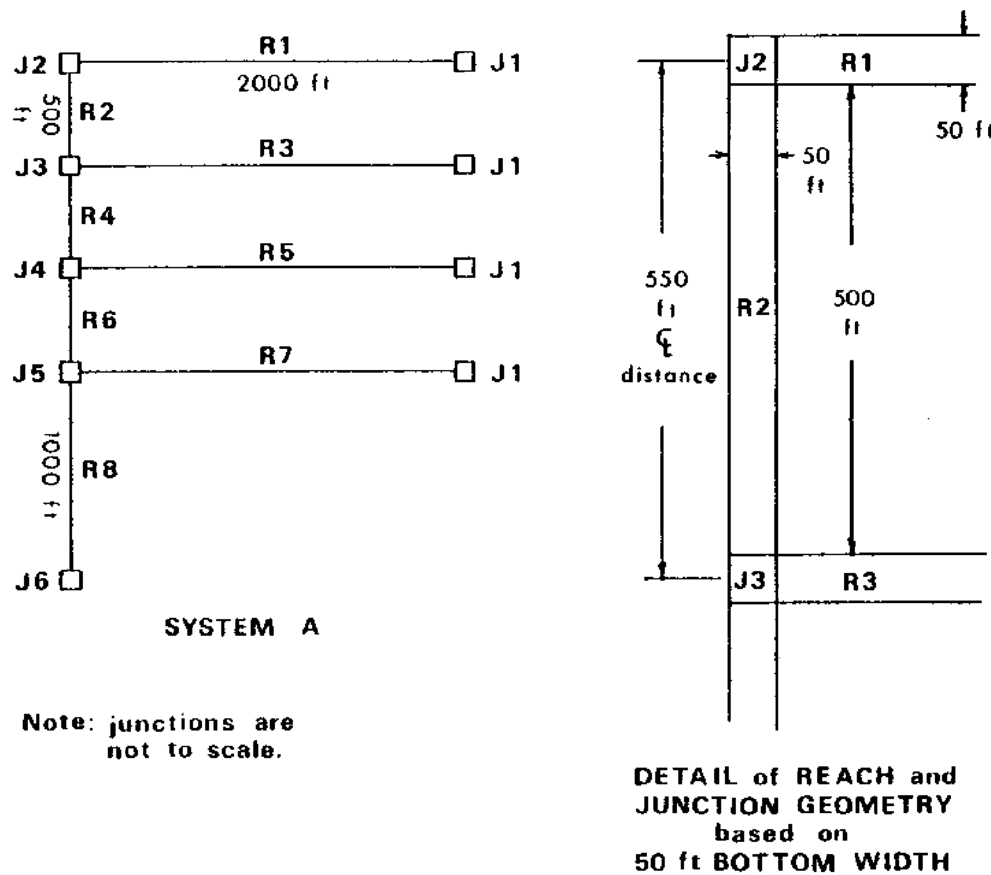


Figure 13.11 - Layout and Dimensions of System A.

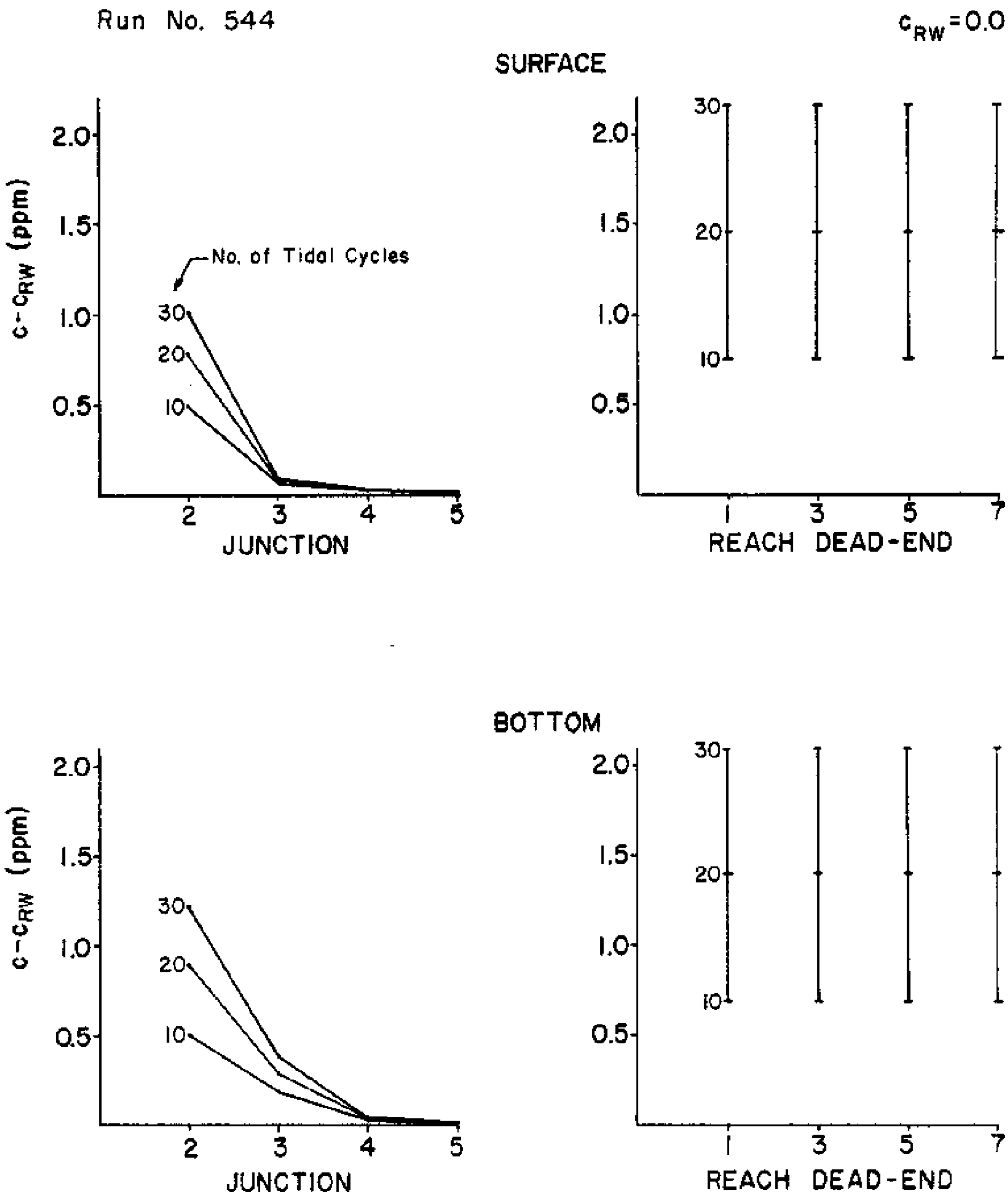


Figure 13.12 - Values of Surface and Bottom Concentrations for No Wind at Junctions and Dead-Ends at High Tide After Thirty Tidal Cycles, System A.

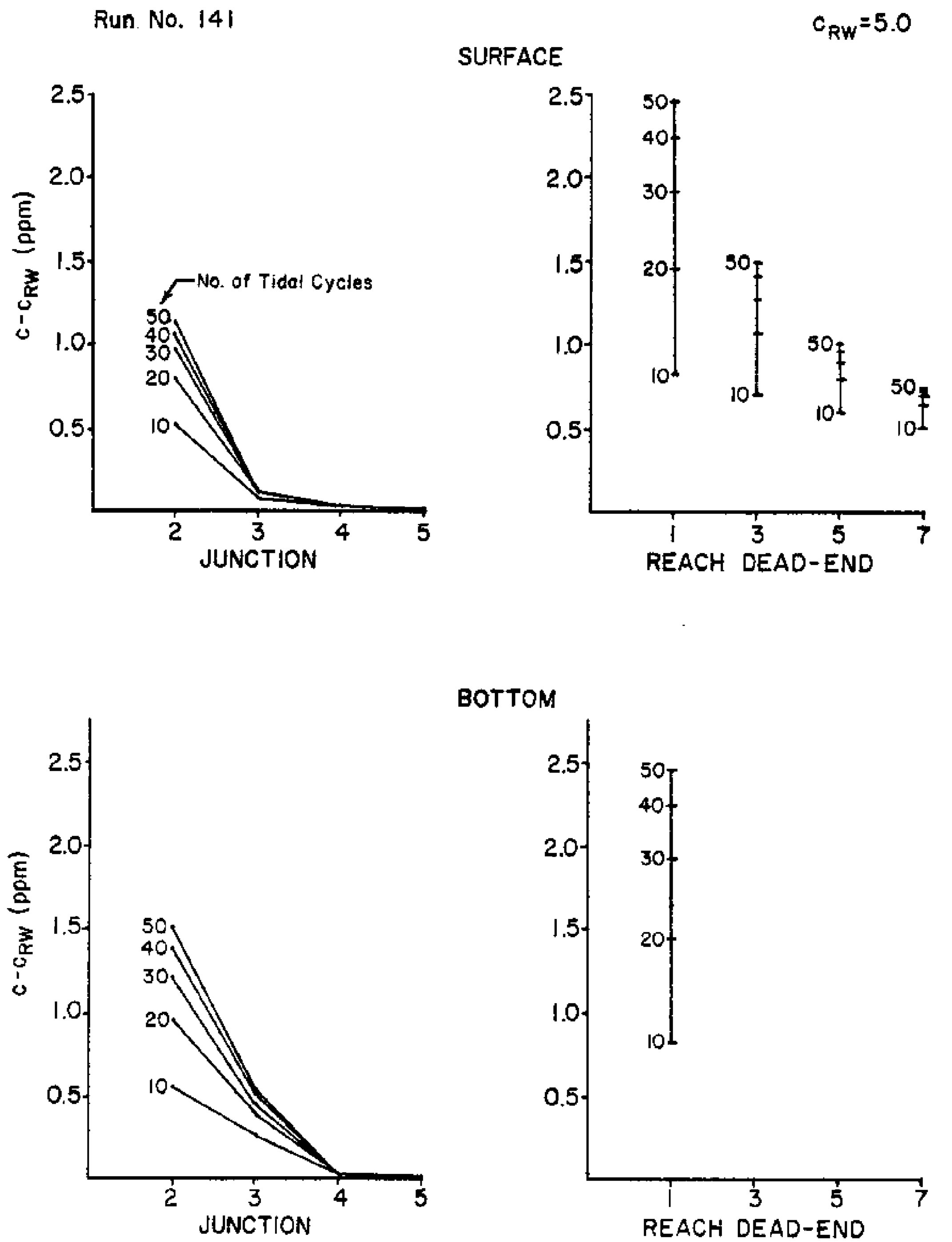


Figure 13.13 - Values of Surface and Bottom Concentration for West Wind, at Junctions and Dead-ends at High Tide After Fifty Tidal Cycles, System A.

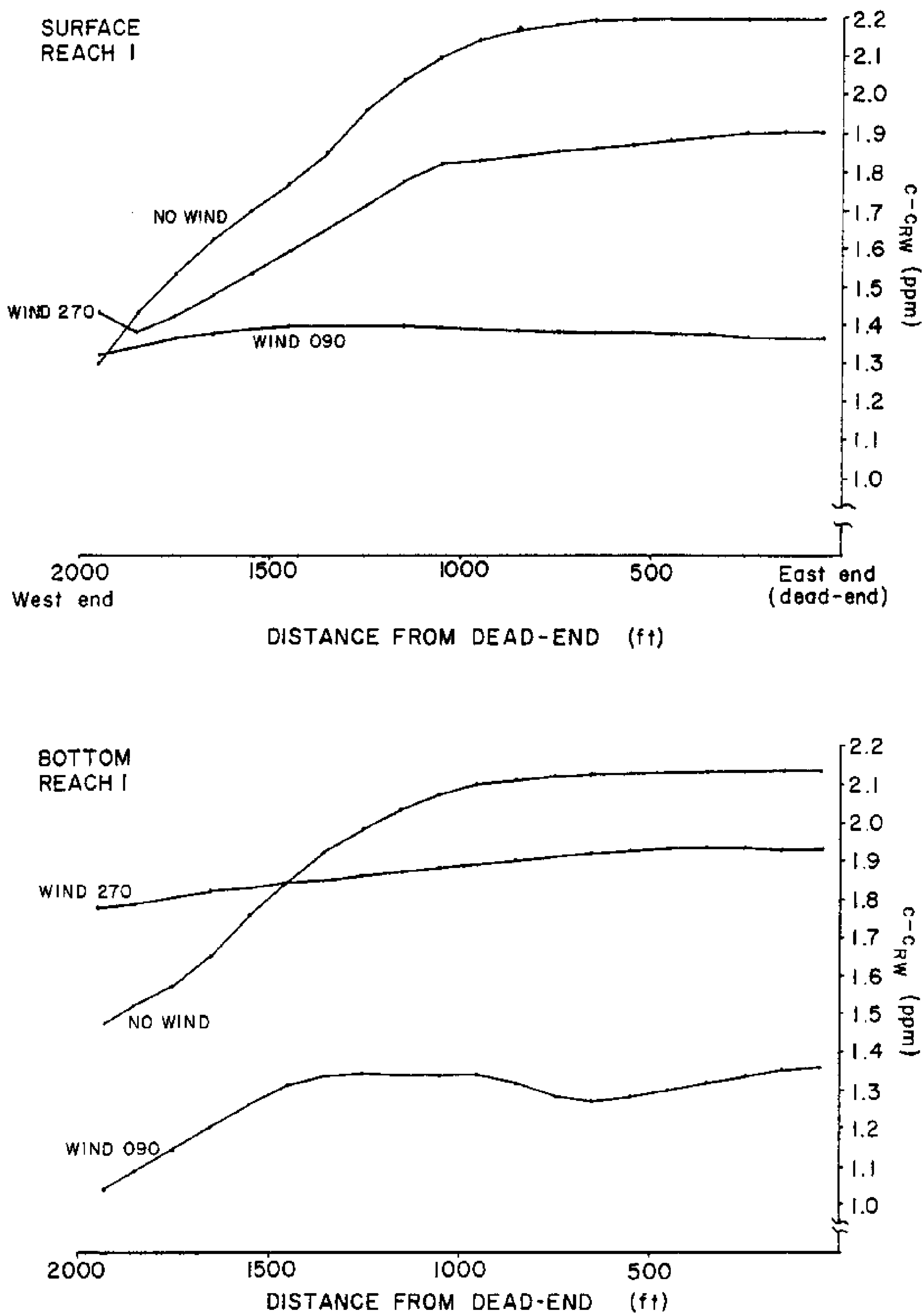


Figure 13.14 - Values of Surface and Bottom Concentrations for Three Wind Conditions in Reach Number 1 at High Tide After Thirty Tidal Cycles, System A.

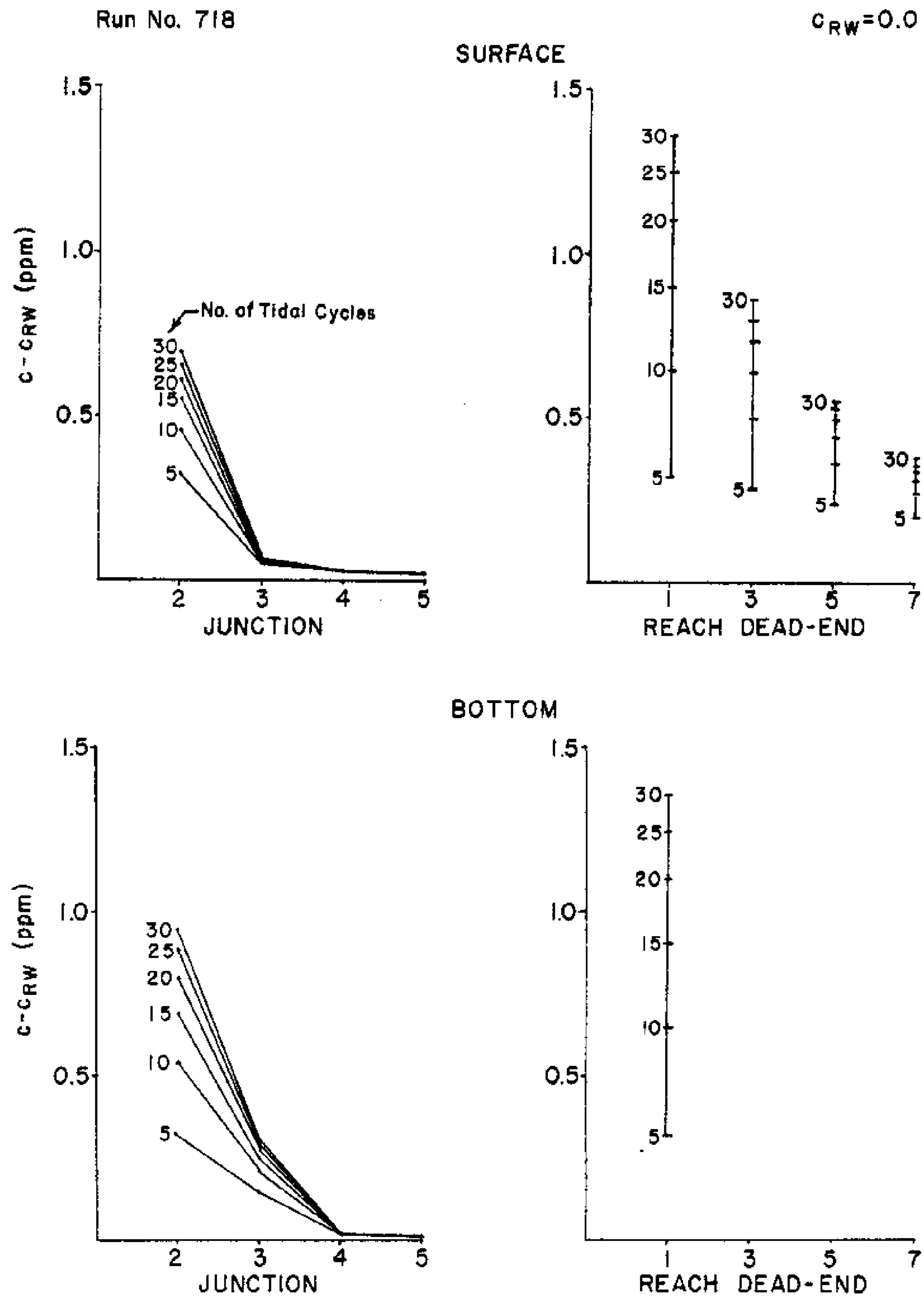


Figure 13.15 - Values of Surface and Bottom Concentration for East Wind, at Junctions and Dead-ends at High Tide After Thirty Tidal Cycles, System A.

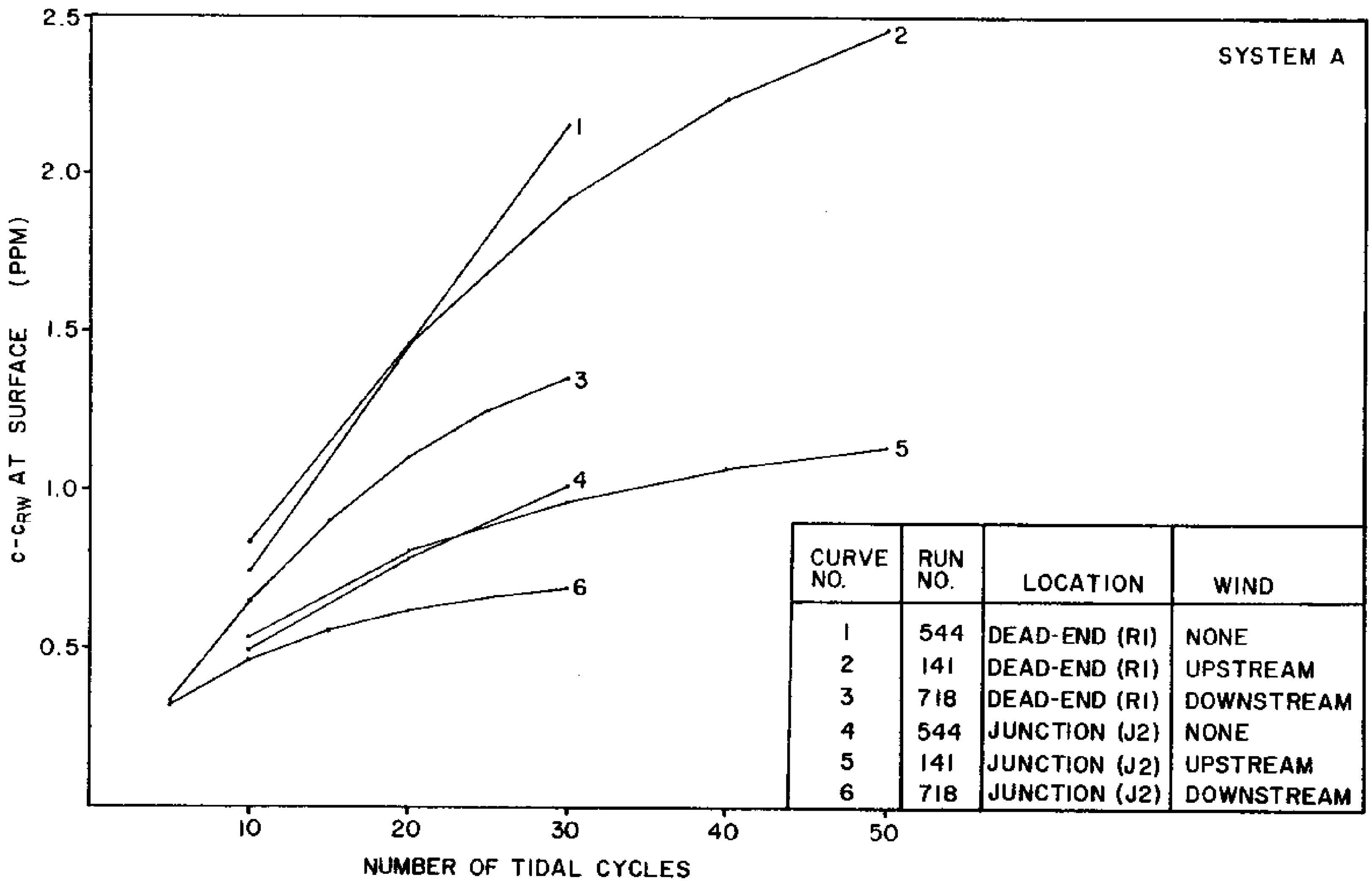
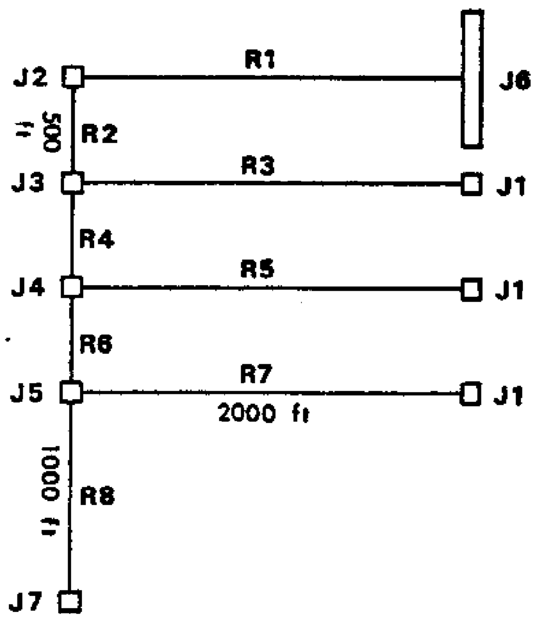


Figure 13.16 - Values of Surface Concentrations for Three Wind Conditions at Dead-End of Reach Number 1 and Junction 2 vs. Number of Tidal Cycles from Beginning of Simulation, System A.



SYSTEM B

Figure 13.17 - Layout and Dimensions of System B.

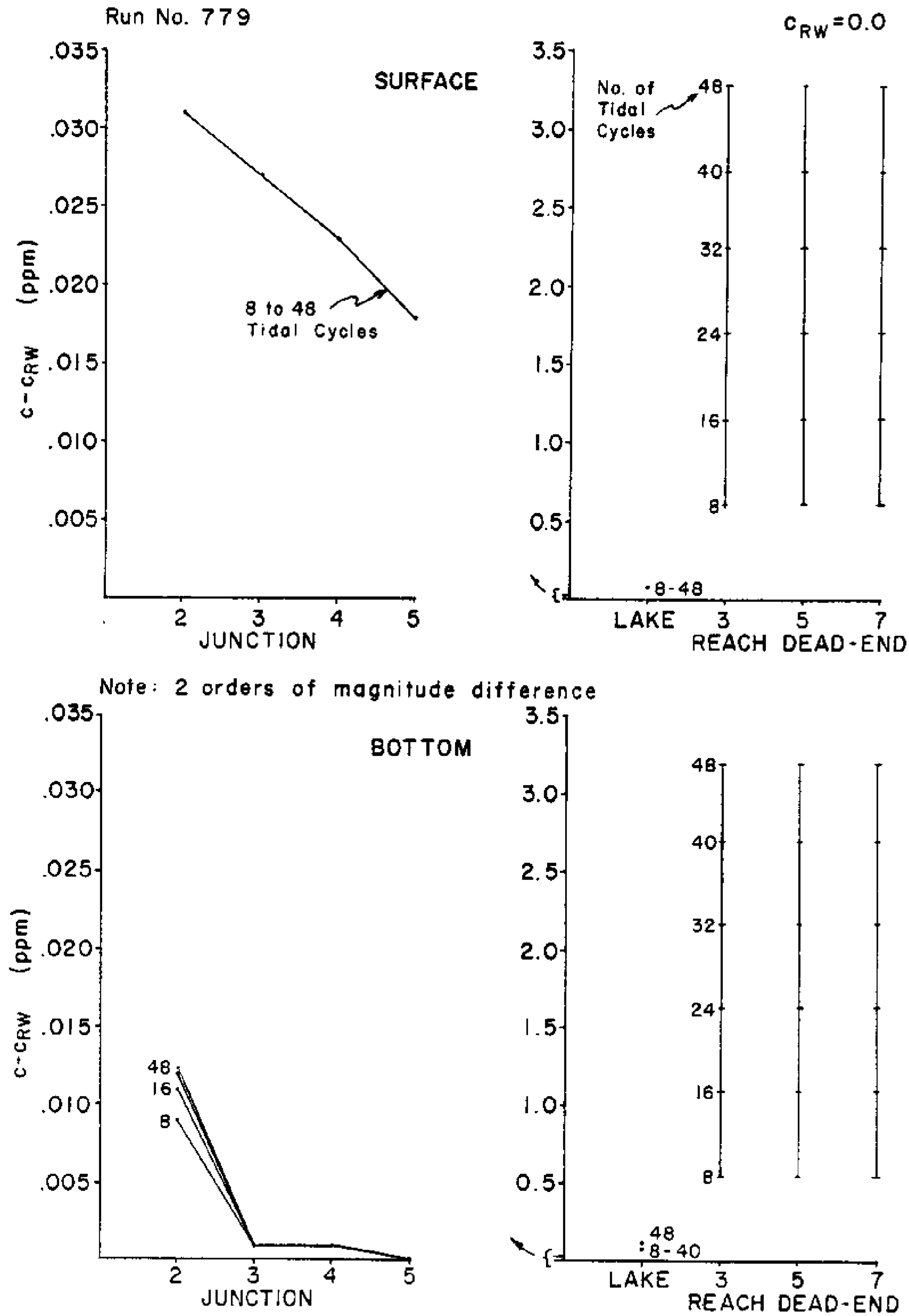


Figure 13.18 - Values of Surface and Bottom Concentrations for No Wind at Junctions and Dead-ends at High Tide After Forty-eight Tidal Cycles, System B.

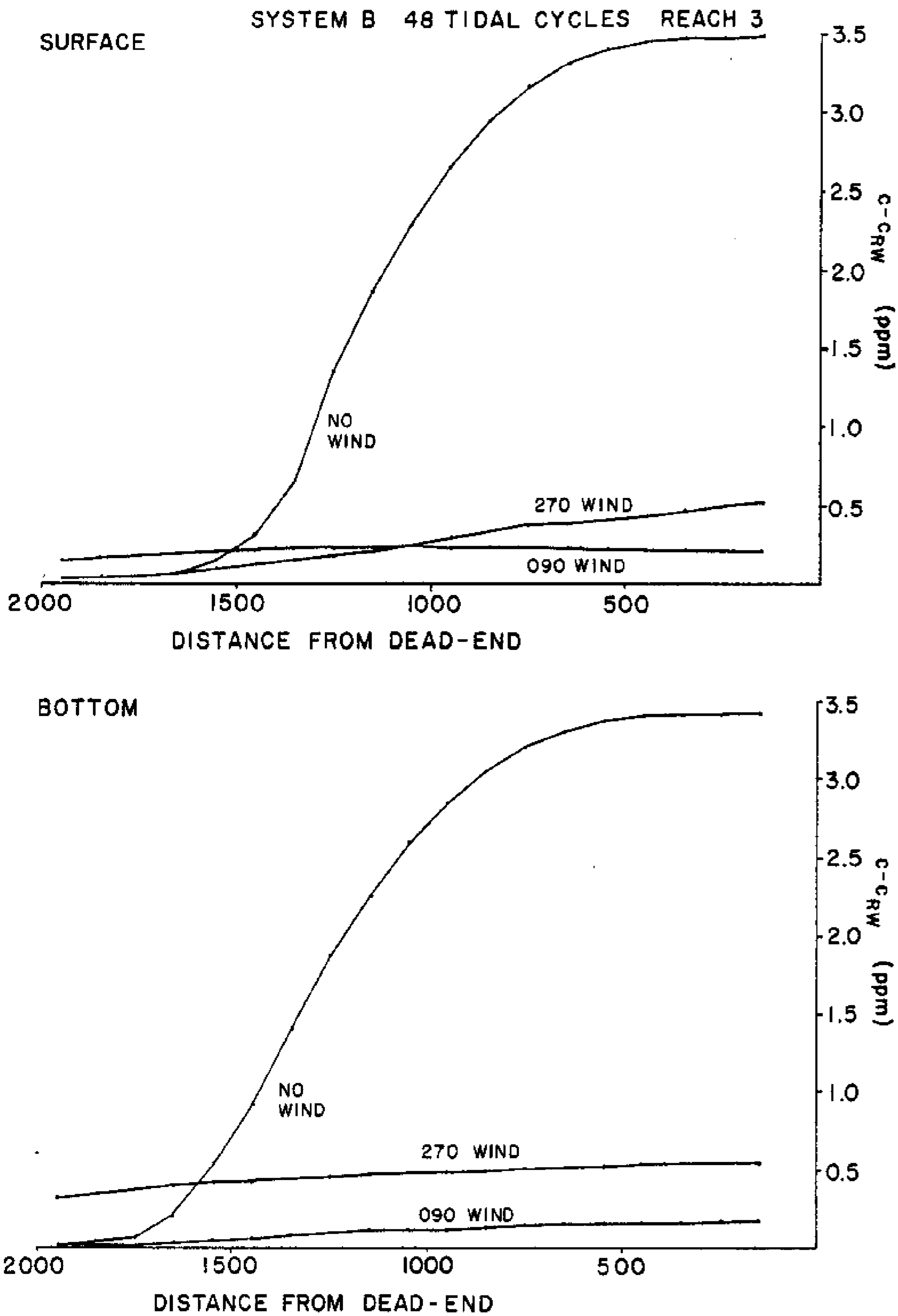


Figure 13.19 - Values of Surface and Bottom Concentrations for Three Wind Conditions in Reach Number 3 at High Tide After Forty-eight Tidal Cycles, System B.

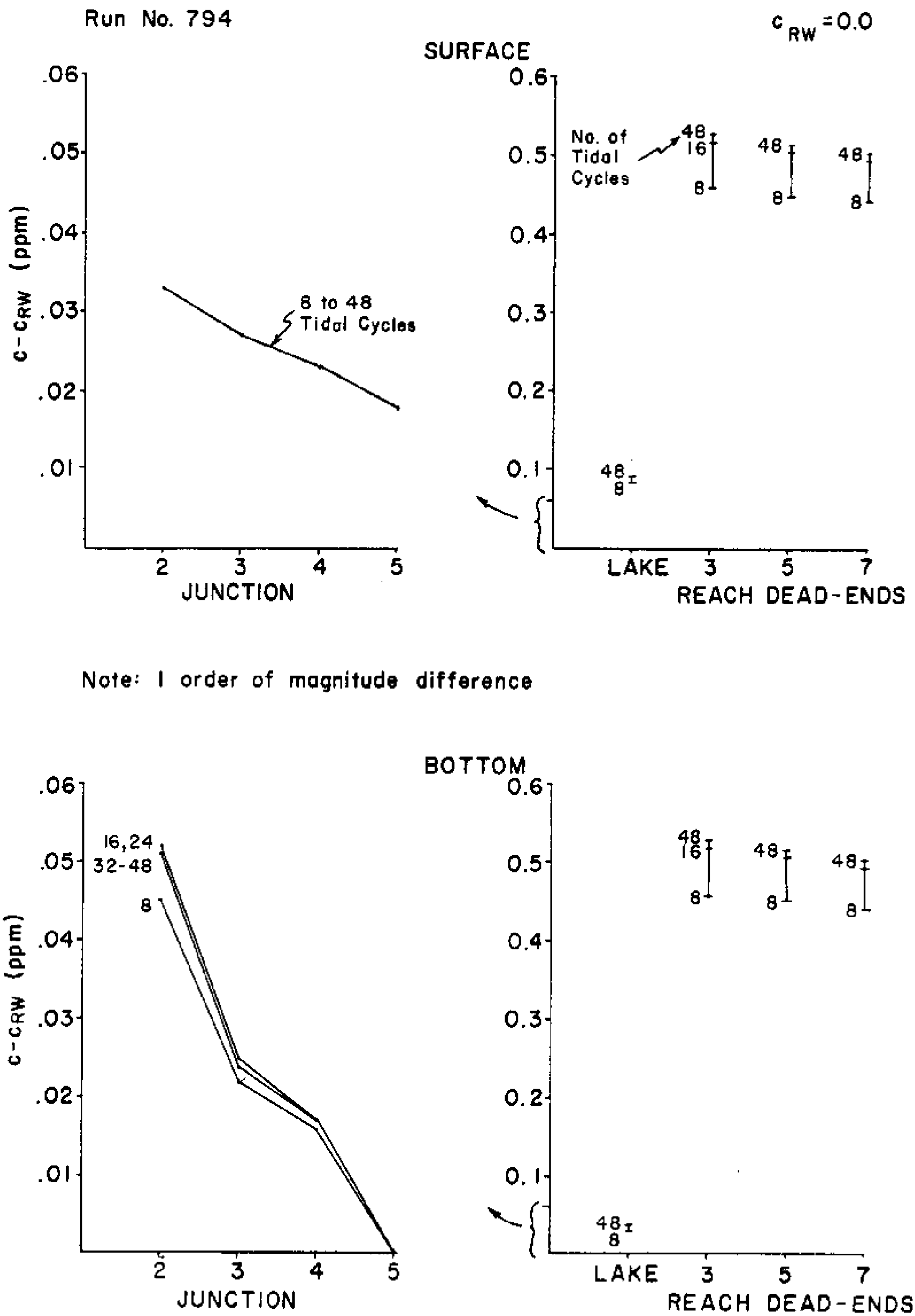


Figure 13.20 - Values of Surface and Bottom Concentrations for West Wind at Junctions and Dead-ends at High Tide After Forty-eight Tidal Cycles, System B.

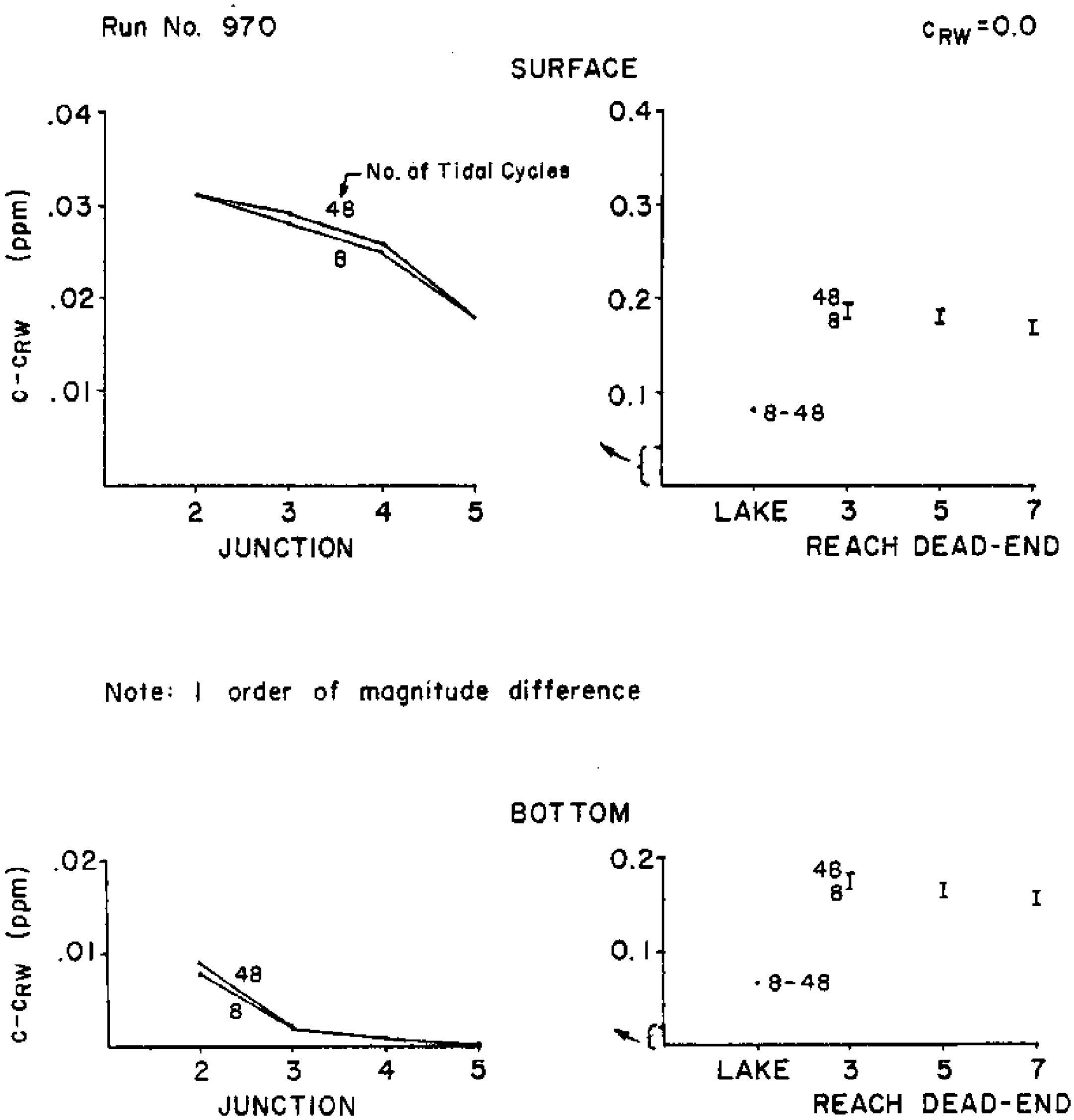


Figure 13.21 - Values of Surface and Bottom Concentrations for East Wind at Junctions and Dead-ends at High Tide After Forty-eight Tidal Cycles, System B.

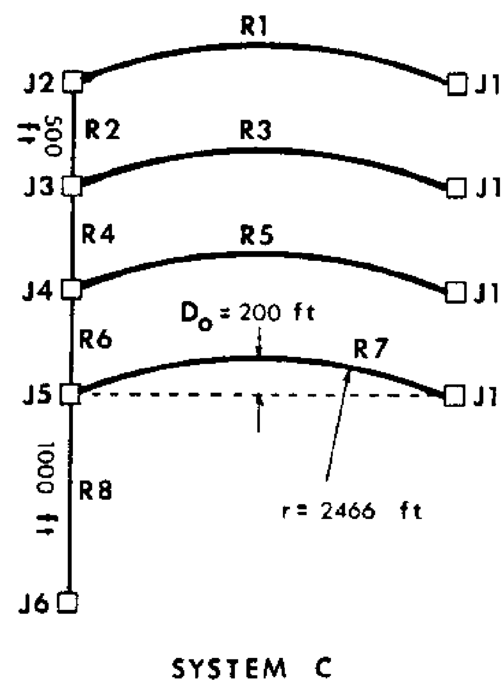


Figure 13.22 - Layout and Dimensions of System C.

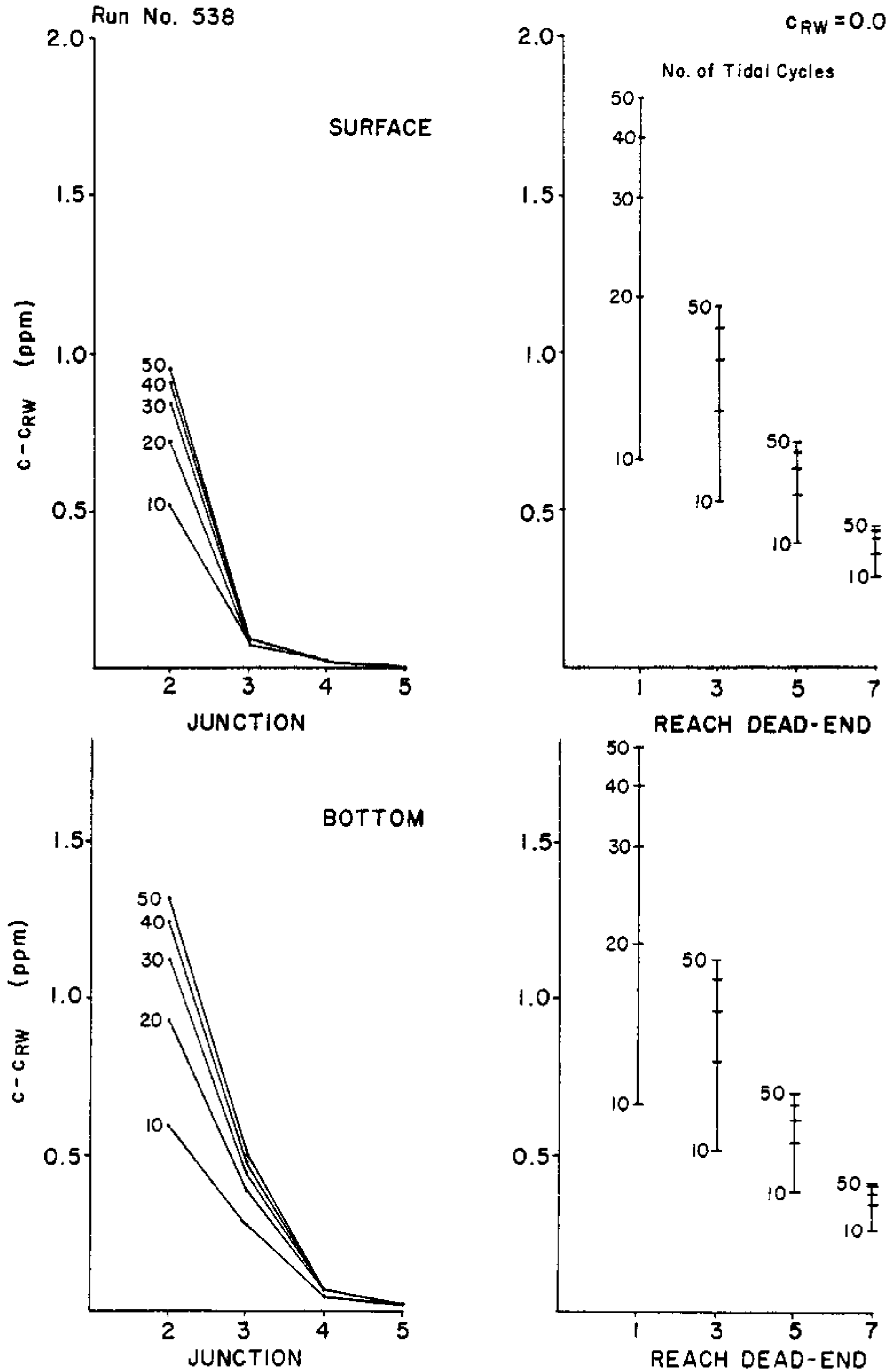


Figure 13.23 - Values of Surface and Bottom Concentrations for Upstream Wind, at Junctions and Dead-ends at High Tide After Fifty Tidal Cycles, System C.

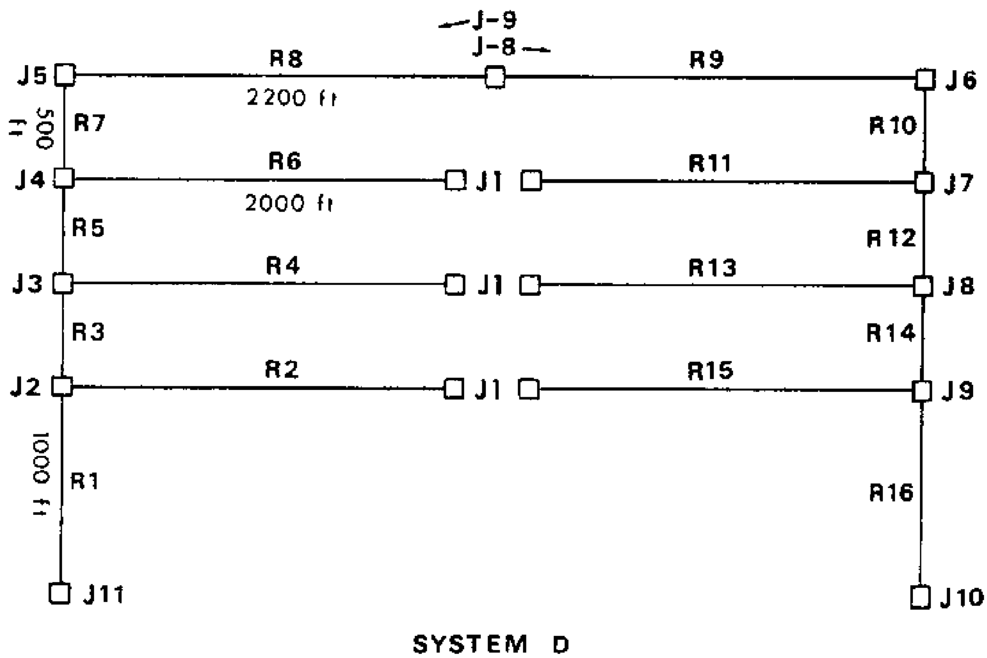


Figure 13.24 - Layout and Dimensions of System D.

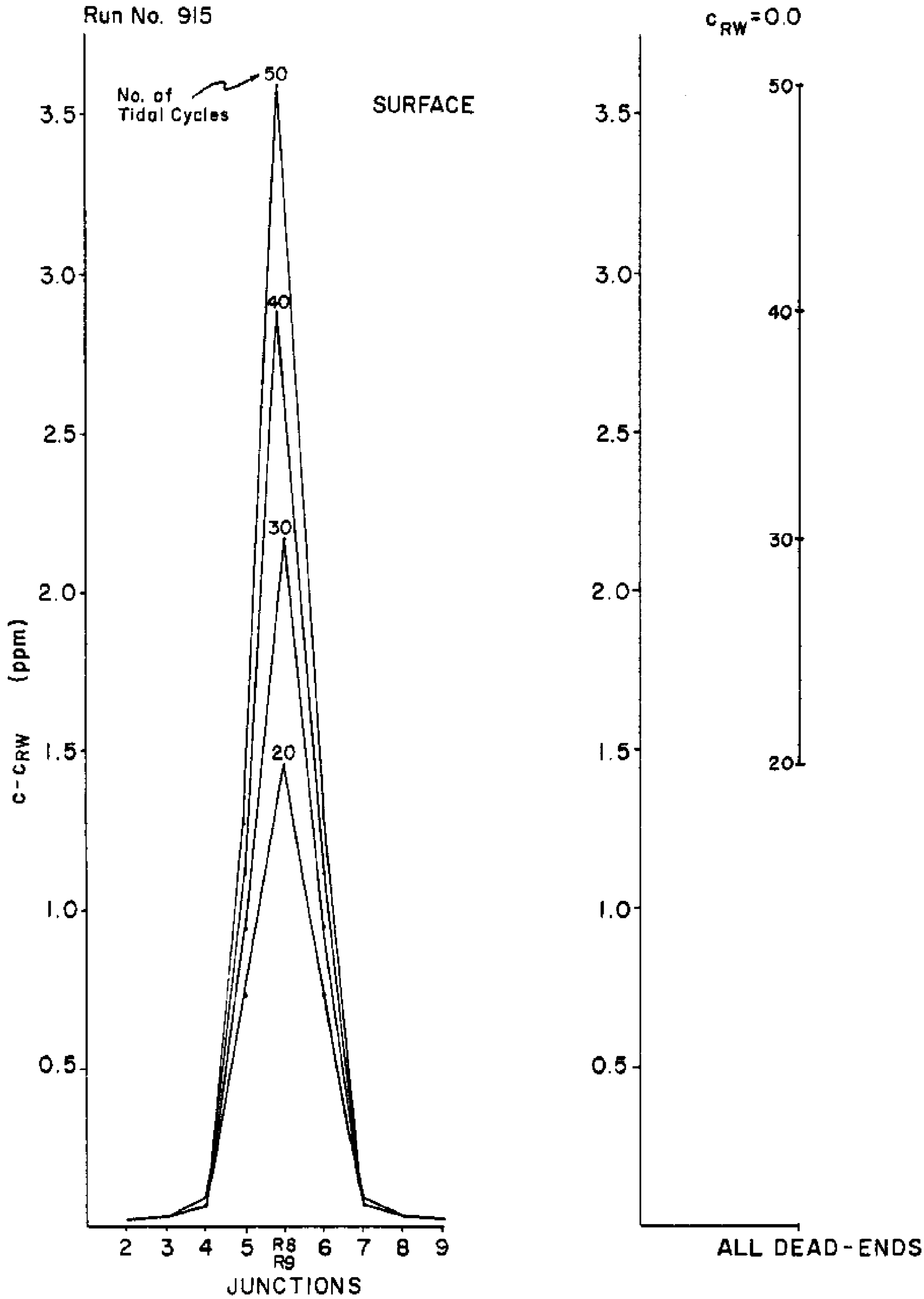


Figure 13.25a - Values of Surface Concentrations for No Wind at Junctions and Dead-ends at High Tide After Fifty Tidal Cycles, System D.

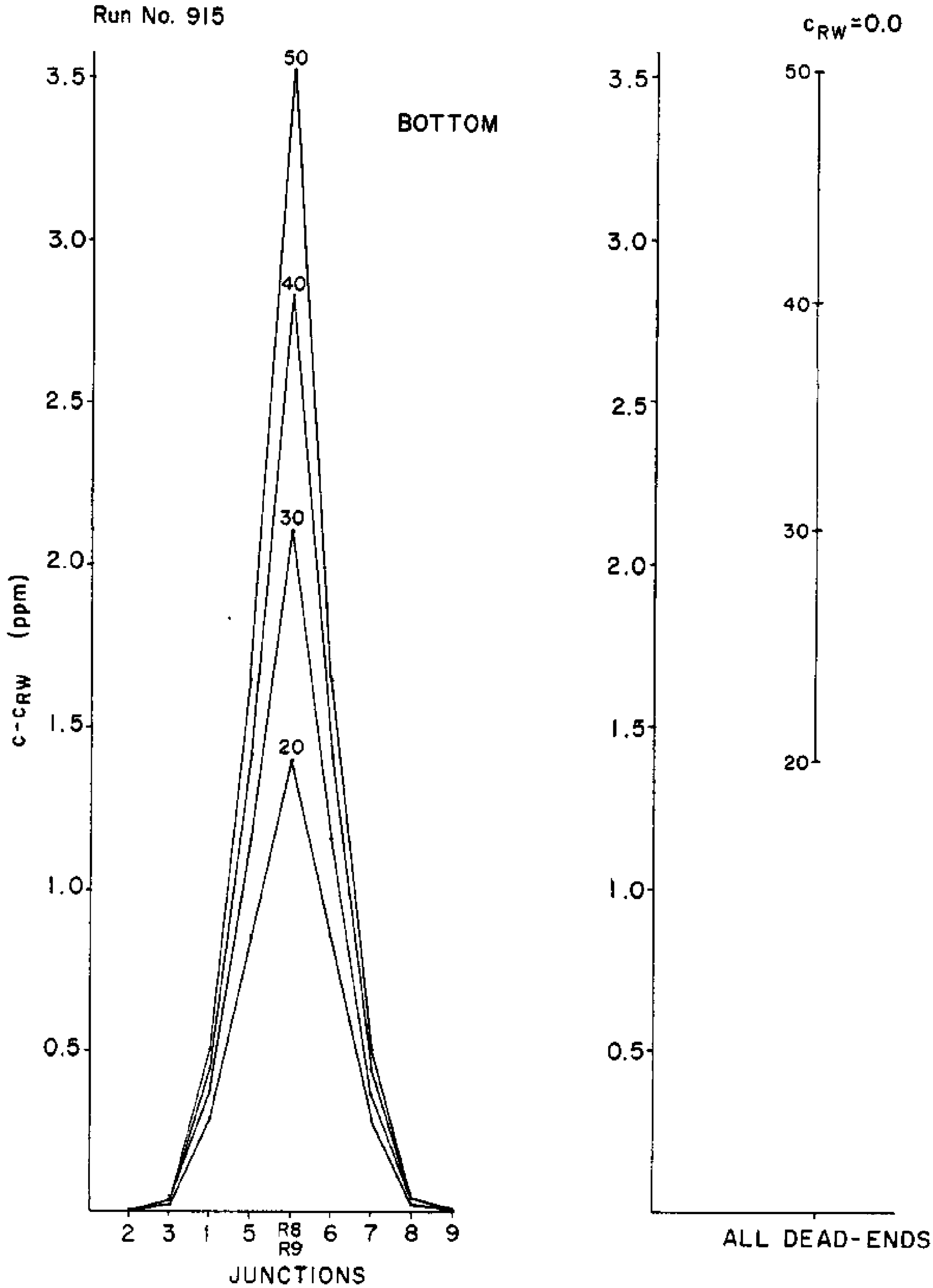


Figure 13.25b - Values of Bottom Concentrations for No Wind at Junctions and Dead-ends at High Tide After Fifty Tidal Cycles, System D.

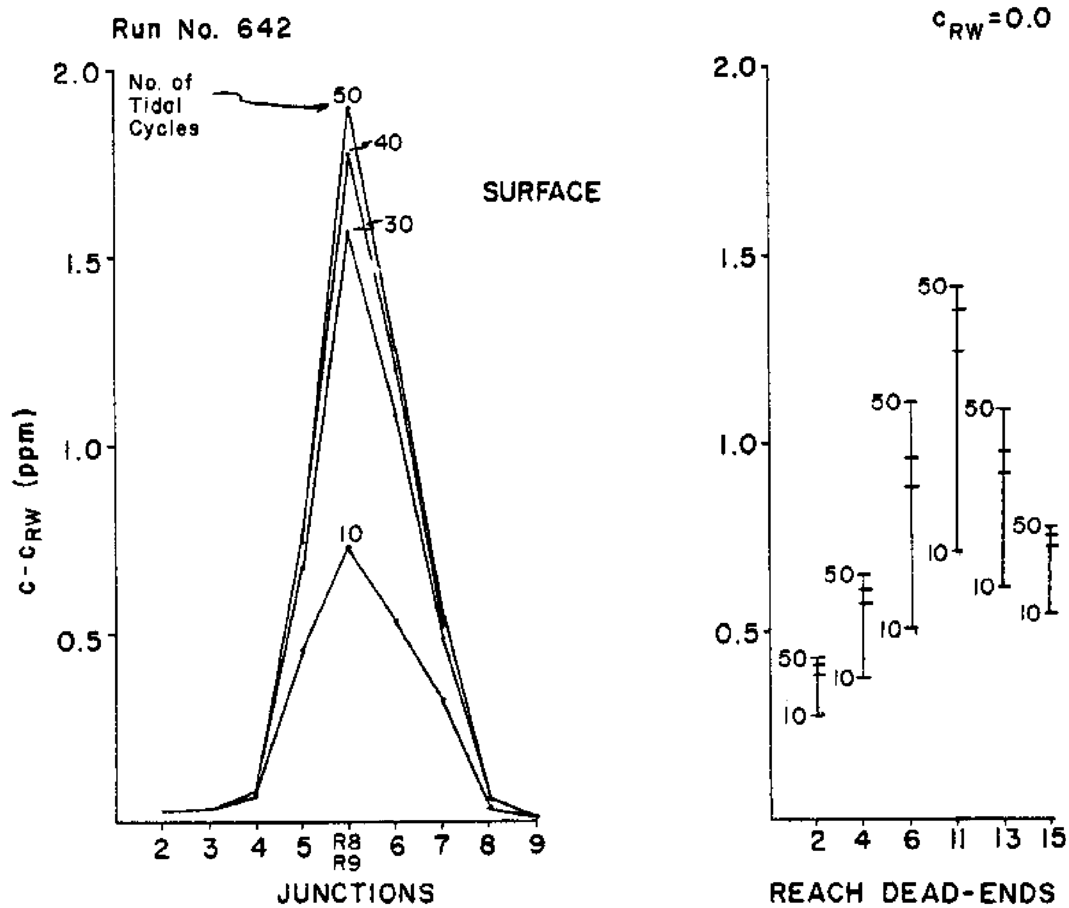


Figure 13.26a - Values of Surface Concentrations for East Wind, at Junctions and Dead-ends at High Tide After Fifty Tidal Cycles, System D.

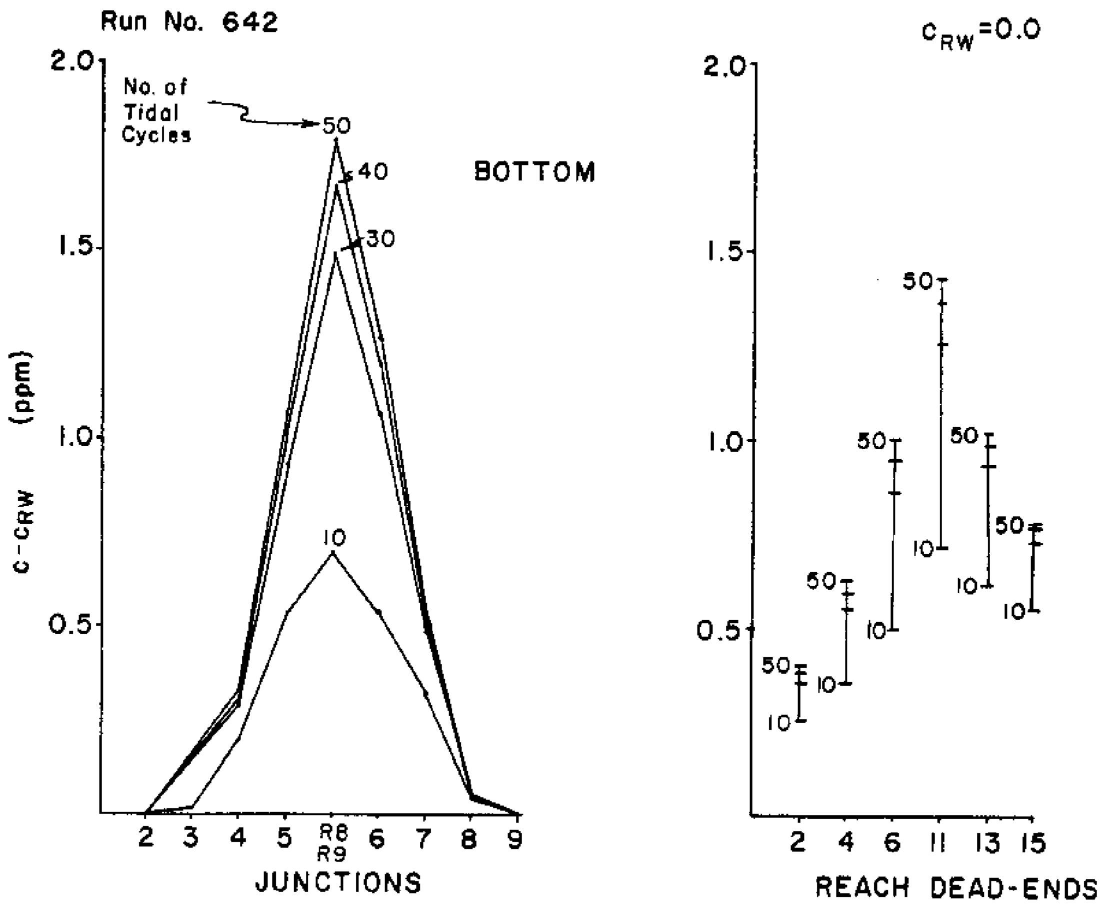


Figure 13.26b - Values of Bottom Concentrations for East Wind, at Junctions and Dead-ends at High Tide After Fifty Tidal Cycles, System D.

Table 13.1 - 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 13.1 - 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.
	Inter-tidal 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 13.1 - 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 13.2 - 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 ± 10° with respect to prevailing wind if possible.
Lagoons and Basins	Maximum size specified.
Channel Geometry	Orient axis of low-flushing channel ± 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 13.3 - 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 Bottom Width : 40 ft Surface Width: 100 ft Trial values to be adjusted for stability

Table 13.4 - Bed and Bank Samples, 57 Acres Canal System

Bed Samples

Sample Location	$d_e = d_{35\%}$ (mm)	M.I.T. Description	Specific Weight of Submerged Solid $\gamma_s - \gamma$ (Lb/Ft^3)	Plasticity Index PI=LL-PL	Critical Horizontal Shear Stress $\tau_{cr.h}$ (psf)
A	0.157	Fine Sand	92.0		0.0027
B	0.155	Fine Sand	92.0		0.0026
C	0.167	Fine Sand	93.9		0.0029
D	0.150	Fine Sand	88.9		0.0025
E	0.00072	Medium Clay	64.5	72	0.0700
F	0.00018	Fine Clay	70.8	114	0.0900
G	0.0073	Medium Silt	67.7	72	0.0200
H	0.175	Fine Sand	73.3		0.0024
I	0.145	Fine Sand	85.8		0.0023
J	0.147	Fine Sand	85.8		0.0023
K	0.001	Coarse Clay	48.3	108	0.0600
L	0.00024	Medium Clay	50.8	98	0.0700
M	0.114	Fine Sand	75.2		0.0016
N	0.150	Fine Sand	88.9		0.0025
O	0.300	Medium Sand	101.4		0.0026
P	0.151	Fine Sand	88.9		0.0025

Bank Samples

Sample Location	$d_e = d_{35\%}$ (mm)	M.I.T. Description	Specific Weight of Submerged Solid $\gamma_s - \gamma$ (lb/ft^3)	Critical Horizontal Shear Stress $\tau_{cr.h}$ (psf)
1	0.213	Medium Sand	92.6	0.0036
2	0.202	Medium Sand	91.5	0.0034
3	0.190	Fine Sand	90.3	0.0032
4	0.205	Medium Sand	91.8	0.0035
5	0.916	Fine Sand	90.9	0.0033
6	0.221	Medium Sand	93.4	0.0038
7	0.235	Medium Sand	94.9	0.0041
8	0.181	Fine Sand	89.4	0.0030
9	0.200	Medium Sand	91.3	0.0034
10	0.213	Medium Sand	92.6	0.0036

Table 13.5 - Calculations of Spatial Mean Velocity, Bed Shear Velocity, Bed Shear Stress, and Nikuradse's k from logarithmic Velocity Profile, 57 Acres Canal System.

Location (See Figure 3 and Appendix B)	Spacial Mean Velocity $\bar{u}$ (fps)	Bed Shear Velocity u^* (fps)	Bed Shear Stress τ_o (psf)	Nikuradses k (ft)
EX5	-0.006	0.016	0.0005	1.1
KU1	0.074	0.042	0.0034	18.6
KU2	0.078	0.123	0.0293	34.7
KU4	0.114	0.049	0.0047	22.1
KU6	-0.054	0.065	0.0082	24.7
KU8	-0.040	0.045	0.0039	31.1
KU9	-0.057	0.046	0.0042	9.0
M3	0.242	0.038	0.0029	8.2
M4	0.213	0.038	0.0029	15.0
M5	0.164	0.023	0.0011	17.5
N2	-0.310	0.138	0.0379	31.2
N3	-0.355	0.094	0.0176	7.8
N4	-0.267	0.056	0.0062	9.8
N5	-0.176	0.042	0.0035	28.6
QR1	-0.556	0.067	0.0089	1.1
QR2	-0.412	0.085	0.0144	6.1
QR5	0.351	0.105	0.0199	35.2
QR6	0.229	0.049	0.0048	43.6
QR7	0.351	0.038	0.0029	25.3
ST4	-0.096	0.089	0.0158	33.8
ST5	-0.247	0.023	0.0011	27.2
ST6	-0.381	0.025	0.0012	30.2
VW3	-0.213	0.139	0.0384	35.4
VW4	-0.227	0.031	0.0019	13.4
VW5	-0.153	0.071	0.0098	13.4
VW10	0.020	0.047	0.0043	4.5
VW11	0.112	0.054	0.0057	5.9
Y2	0.124	0.015	0.0005	13.2

Table 13.6 - 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_0, sq ft/sec	0.0005
Vertical momentum transfer, N_z, sq ft/sec	0.002
Tidal entrance time decay, hr	1.0

Table 13.6 - 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 13.7 - 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			
		Syst. A		Syst. D		Syst. A		Syst. 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 13.8 - 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	13.12
A	None	None	1	50	West	141	13.13
A	None	None	1	30	East	718	13.15
B	100,000	None	1	48	None	779	13.18
B	100,000	None	1	48	West	794	13.20
B	100,000	None	1	48	East	970	13.21
C	None	2466	1	50	East	538	13.23
D	None	None	2	50	None	915	13.25
D	None	None	2	50	East	642	13.26

CHAPTER 14

DESIGN ALTERNATIVES AND EXAMPLE OF MODIFICATION OF AN EXISTING CANAL DESIGN

14.1 Introduction to the Example

In order to illustrate a method by which an existing canal design can be iterated toward a final design which meets the established design criteria and constraints, a *simplified* network will be simulated in this chapter. The design has been constructed by combining several of the comb-structured canal elements described in Section 13.5. The network has been purposely simplified so that comparisions of concentration profiles in various portions of the network can be easily made, and so that the effects of changes in design can be isolated and discussed. Bends will not be included in the design in order 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.

14.2 Description of Existing Canal System

Figure 14.1 is a diagram of the "existing" canal system which has been developed for this example. The site has 5000 ft of tidal shoreline and a depth of 5000 ft to the main road, an area of 573.9 acres. A small pond, six acres in surface area, lies near the main road. Water supply and sewerage and storm water connections are offsite.

The walls 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 collection of trash at the downwind ends of finger canals have been experienced. The owner has decided, since there is space available along the canal frontage to provide sloping, vegetated banks for stability, and filter mounds and swales leading to a retention basin, to modify the canal banks. The mean tidal amplitude is 1 ft and the prevailing wind direction during the summer is East.

For purposes of comparing the effects of design modifications on the canal system, it is convenient to label the single set of finger canals the "western" section and the double set of finger canals the "eastern" section. The cross-sections of the canals are trapezoidal, with the same dimensions as used in the network variability tests in Section 13.5 (see Table 13.6) except that the length of all finger canals is 1000 ft.

14.3 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 a special feature that may have a critical 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 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) that has the same volume 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 14.2) 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 13.5.2, 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 14.2). When the lake and its associated reach are added, they are designated J10 and R19. The tidal entrance is labeled junction 10 without the lake, and junction 11 when the lake is included.

Results of all tests on the network are plotted on a common form to facilitate comparison (for example, see Figure 14.3). 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 fifty tidal cycles (621 hours, or over twenty-five days), and all plotted values of concentration are cross-sectionally averaged high tide values. Thus, the degree of flushing after fifty tidal cycles at any location in the network can be readily found from the network plots. The rate of flushing at increments of ten tidal cycles is available from the computer printout.

A simulation of wind-induced flushing in the existing canal network is shown in Figure 14.3. This demonstrates that the system flushes relatively uniformly under this steady wind.

14.4 Simulation Objectives

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

1. Simulation of existing conditions in the canal system itself, if it already exists, or in a similar neighboring canal system, for 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 8.

The network variability tests, (Step 3 above) require the introduction of a substance into the canal network which can be used as an indicator of the effectiveness of flushing in various portions of the system. One procedure that has been found to be effective is to initially set the concentration in all cells in the model to a value of 10, 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 point source concentration. This procedure, however, is only good for observing the flushing rate at one point 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 of a reach in the model.

14.5 Flushing Under No-wind Conditions

Under no-wind conditions the trial network flushes very poorly. Figure 14.4 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 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 14.5). 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 inside halves of the finger canals in the western portion and not at all effective in any part of the double set of finger canals.

14.6 Design Alternatives

14.6.1 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 negligable, but in the western part the downstream wind reduces the concentration at the dead-end of the northernmost finger canal (R12) from 10 to less than 7 ppm (Figure 14.6). 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 14.7). In the case with 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 14.3), 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 14.8). 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 14.9) 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 this particular network configuration and this wind speed and direction, 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 14.10, 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

$$\frac{c}{c_o} = K_o e^{Kt} \tag{14.1}$$

where

c_o = background concentration (dimensionless)

K_o = constant associated with an initial 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,

$$K = - \frac{\ln(c_1/c_2)}{N_2 - N_1} \tag{14.2}$$

and

$$N_{10\%} = N_1 - \frac{\ln c_1}{K} \qquad (14.3)$$

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 14.1.

Figure 14.10 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 thirty through fifty 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 by using Equations (14.2) and (14.3) to 10 percent (c/c_0 = 1). The resulting flushing times are 92 tidal cycles for canal R2 and 136 tidal cycles for canal R12. Similarly, the extrapolated flushing times for the other cases with wind, and one without, are summarized in Figures 14.11 and 14.12 and in Table 14.1.

If the effects of a range of mean depths and steady winds are considered in the context of the results of simulations described thus far, it is evident (from Table 14.1) 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 on the average than the 8 ft depth, to maximize tidal flushing, 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 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.

14.6.2 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 velocites 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 some effect in the downstream portions of the finger canals.

The surface area of the trial canal network is 1,680,000 ft^2. 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 (15 percent of total network area) and 500,000 ft^2. The results for the 8-ft-deep network, no wind, and the smaller lake area, are shown in Figure 14.13. The results of this simulation show that the concentration values in the main channels are reduced approximately 50 percent after fifty tidal cycles, and that the increased flushing extends, with decreasing effect, almost the entire distance into the finger canals. Comparison with Figure 14.4 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 14.14) 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 fifty 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 negligable 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 14.15). Reductions of more than 10 percent at the dead-ends of finger canals are a slight improvement over 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 values after fifty tidal cycles (Figure 14.16). Comparing Figure 14.16 with the results from identical conditions, except for no lake, (Figure 14.7) shows an improvement from 34 percent to 12 percent flushing at the dead-end of canal R2, and a slight but negligable 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 thirty tidal cycles (Figure 14.17).

The cases with lakes are compared by means of flushing curves for bottom concentrations in canal R2 in Figure 14.18. The two cases for the 8-ft-deep network and no wind, both 15 percent and 30 percent lake surface area, do not flush well enough to warrent 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 14.1). 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 obviously, from the simulations presented in this section, the most effective flushing mechanism presented thus far for tidal canal networks.

14.6.3 Comparison of Effects of Steady and Variable Wind

From the comparative tests so far described, 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 fifty 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:

Time	Specified Wind Speed, mph	Specified Wind 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 of October winds in the region south of Jupiter inlet. If the network model described in this section were to be used to simulate a canal system at 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 at the 0300 wind.

An "equivalent" steady wind was defined by noting that the shape of the wind velocity distribution over the 18-hr period during which the variable wind is active is approximately Gaussian. Taking the peak value to be 8 mph, numerical integration at 1-hr intervals over 24 hr 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 a 2 mph wind from 090 were shown in Figure 14.6. The results from a simulation using the variable wind are shown in Figure 14.19. From a comparison of these two figures, it is obvious that the variable wind has provided a great deal more flushing than the steady wind. The most likely explanation is that, when periods of maximum tidal velocites occur simultaneously with intervals of high wind velocity (on the order of 6 to 8 mph) there is a great deal of mixing and movement of pollutant in the system. This activity, even over a relatively short interval of time, is far more effective in flushing the network than the steady 2 mph wind, which has been shown to be very limited.

It may be concluded from the foregoing comparison that the simulation of the transport of substances in a trial canal design should be conducted with variable wind data appropriate for the site. 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, and published data are not transferable to different sites, it follows that the canal designer should take wind measurements at the site over a sufficient period of time either to find a representative wind sequence or to cover the entire length of time to be used in the canal network simulations. The simulations conducted for the evaluation of the hypothetical rectangular canal and the proposed trapezoidal canal are summarized in Table 14.2.

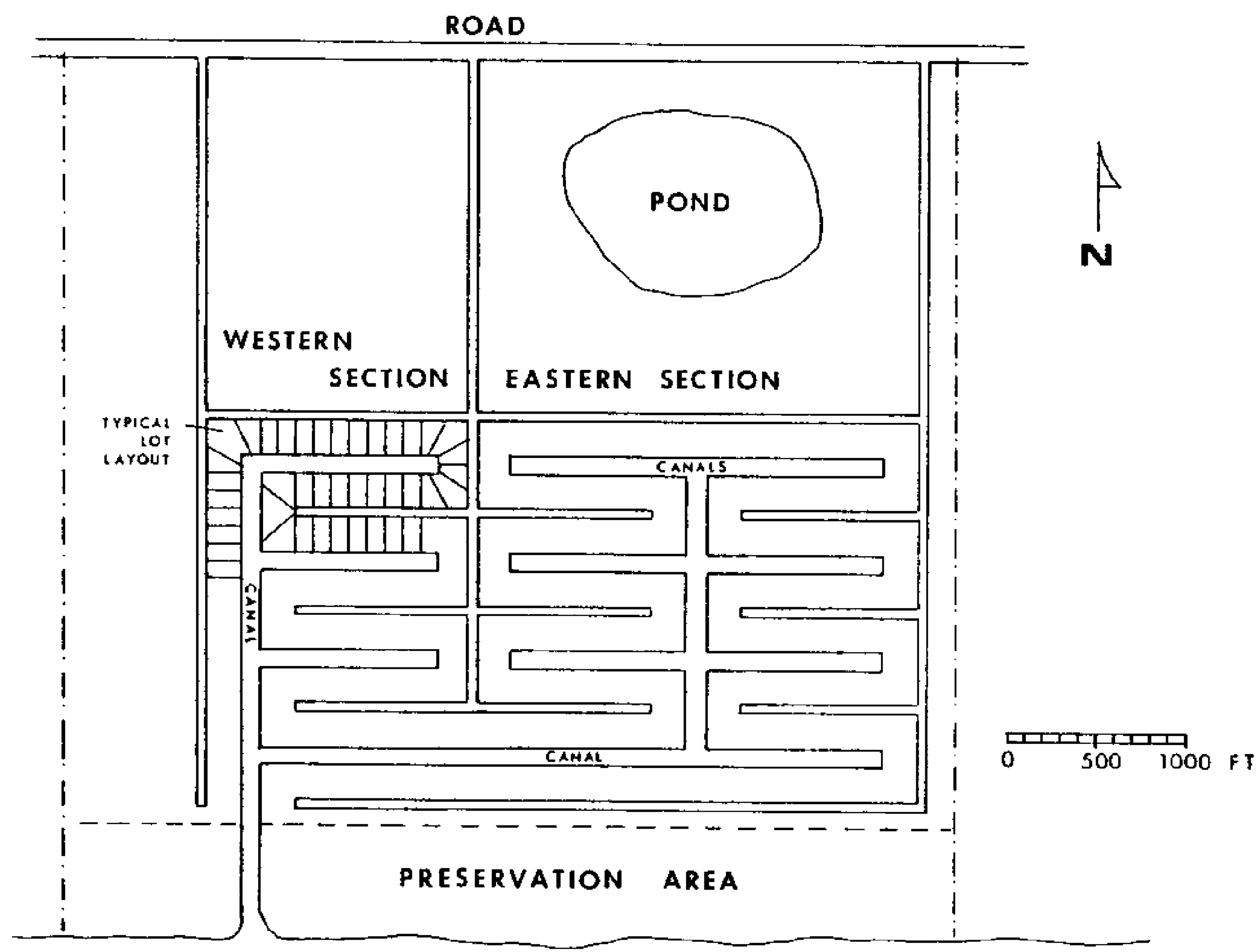
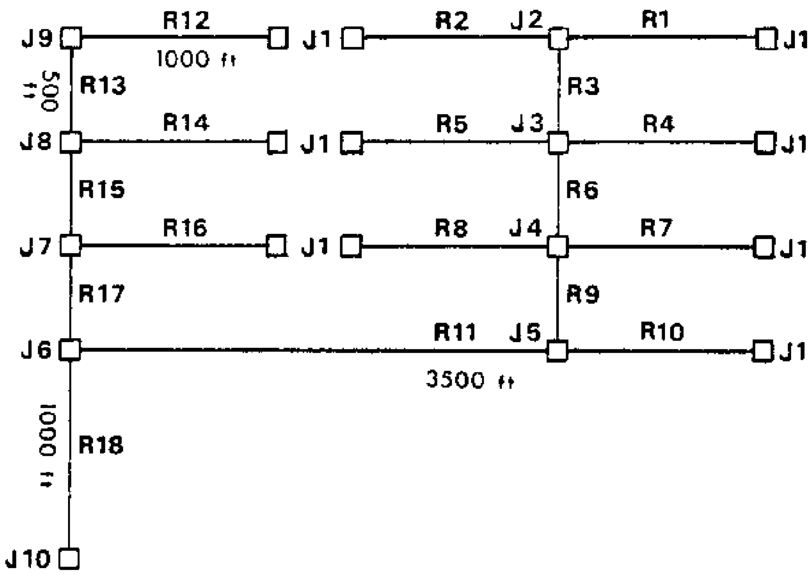
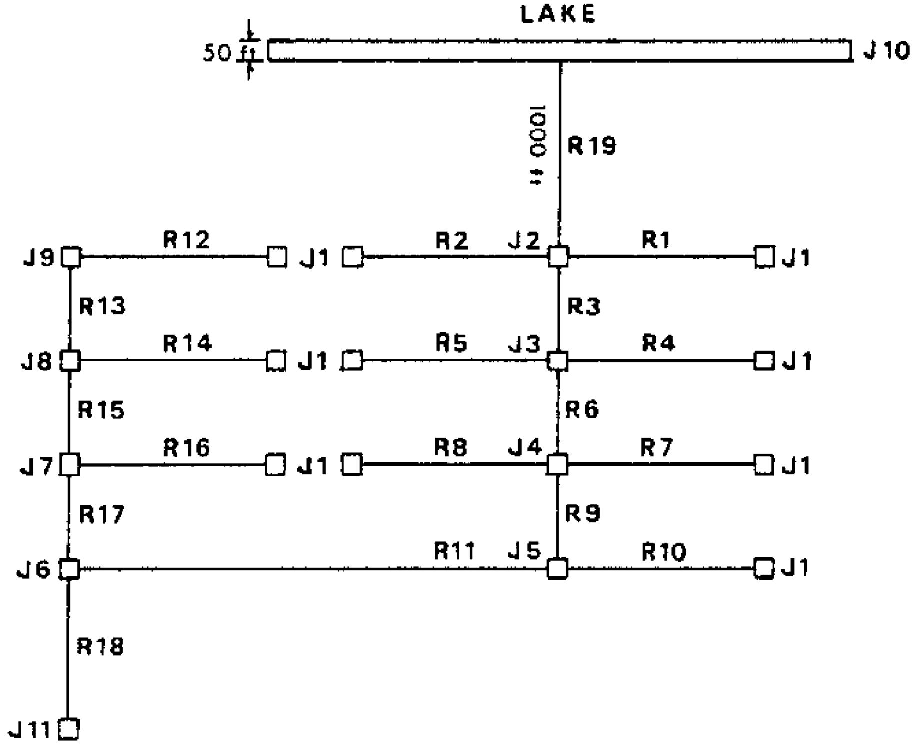


Figure 14.1 - "Existing" Example Canal System.



SYSTEM E

Note: junctions are
not to scale.



SYSTEM EL

Figure 14.2 - Layout of Model Network.

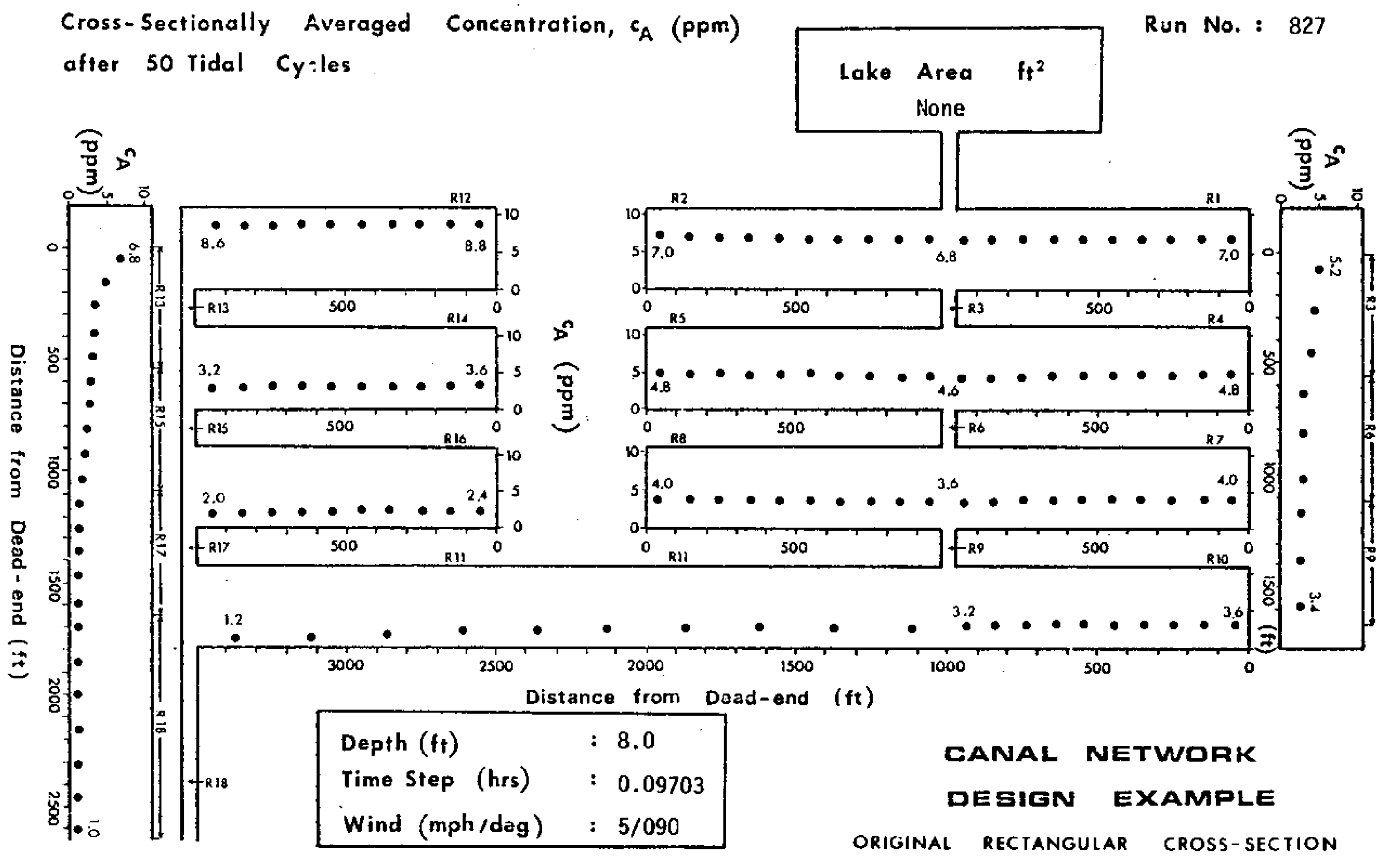


Figure 14.3 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Original Canal Network After Fifty Tidal Cycles, With a Steady Wind of 5 mph from the East (Run No. 827).

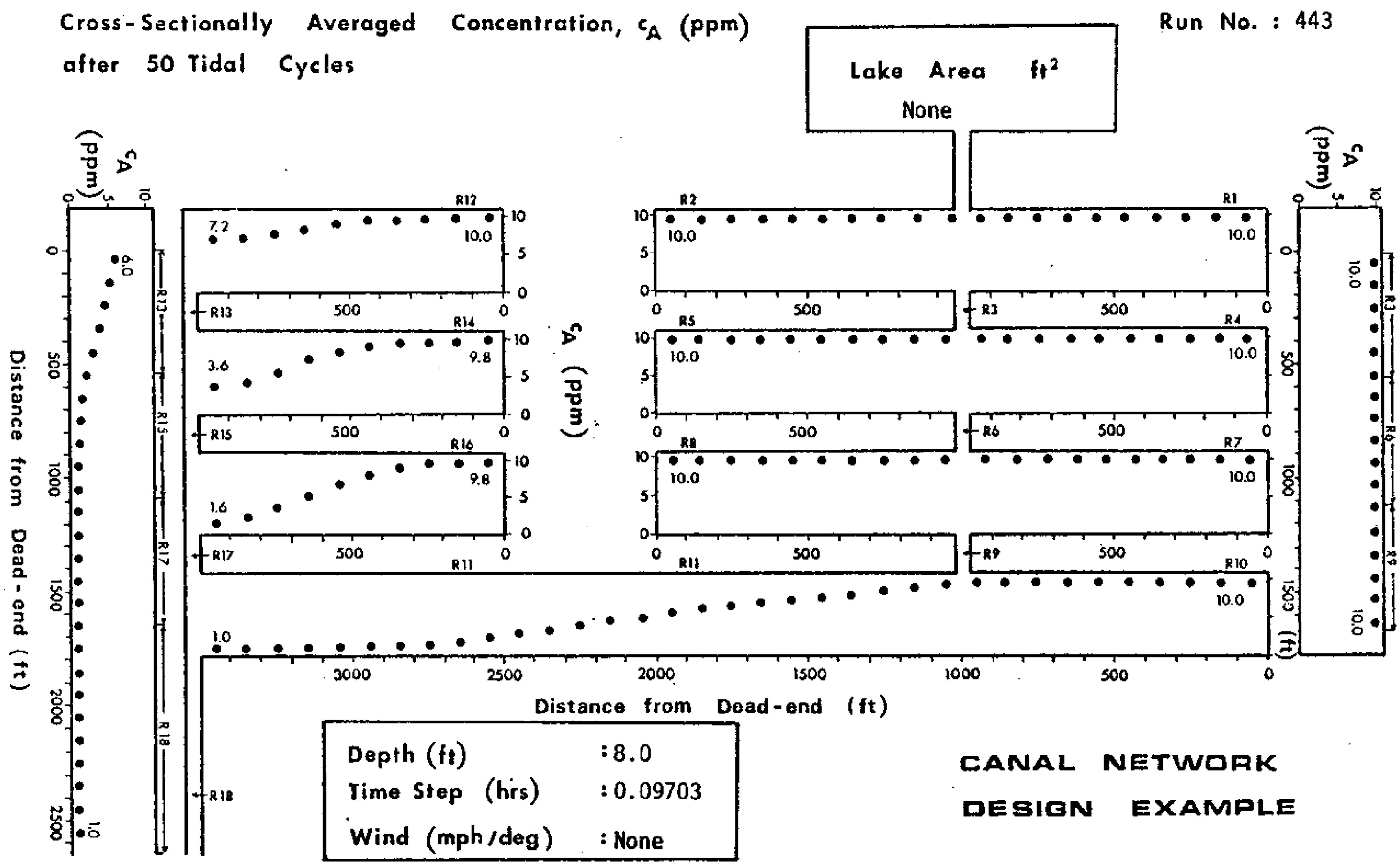


Figure 14.4 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, for No Wind and No Lake (Results of Run No. 443).

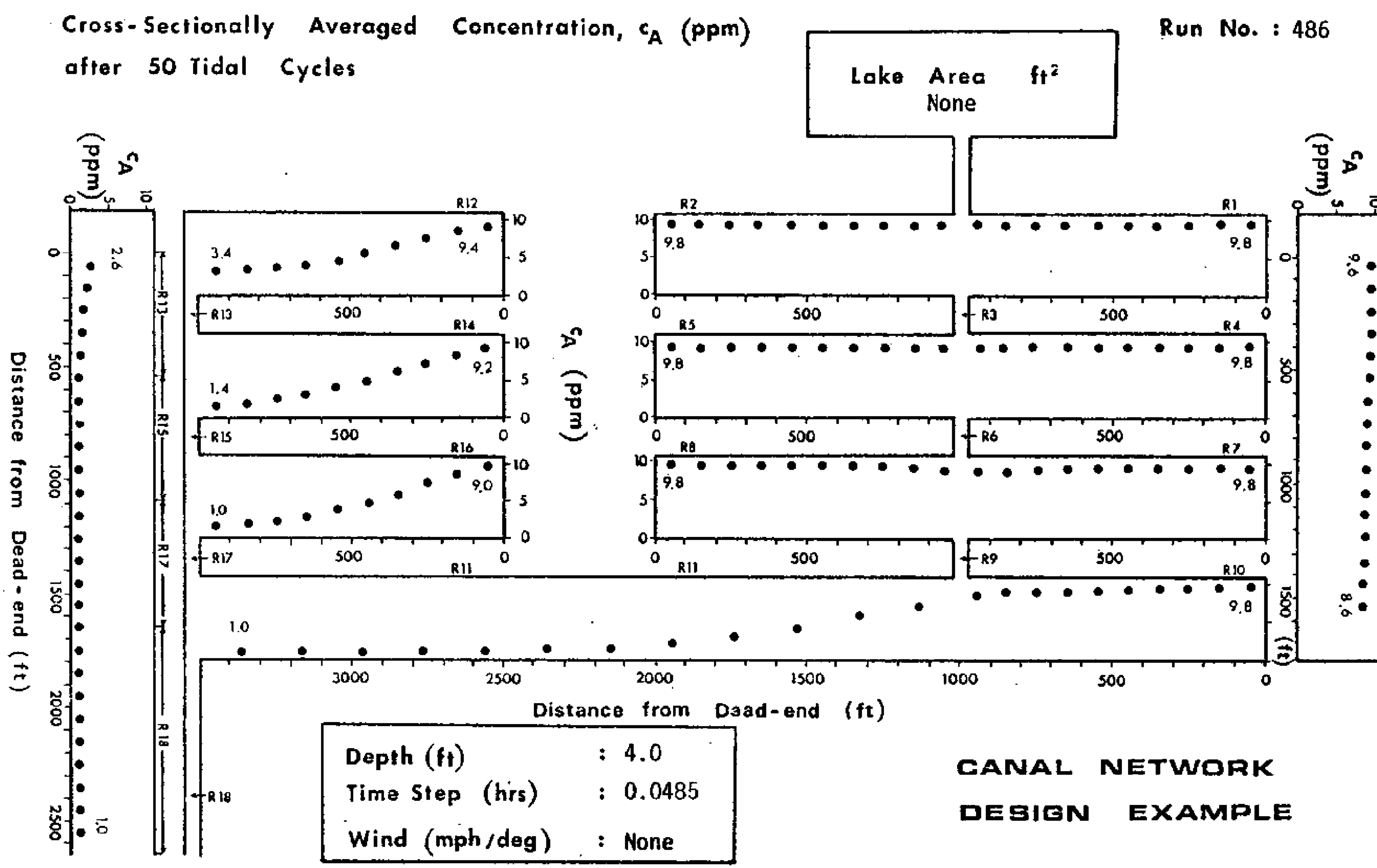


Figure 14.5 - Cross-sectionally Averaged Concentration Profiles in the 4-ft-deep Alternate Canal Network After Fifty Tidal Cycles, for No Wind and No Lake (Results of Run No. 486).

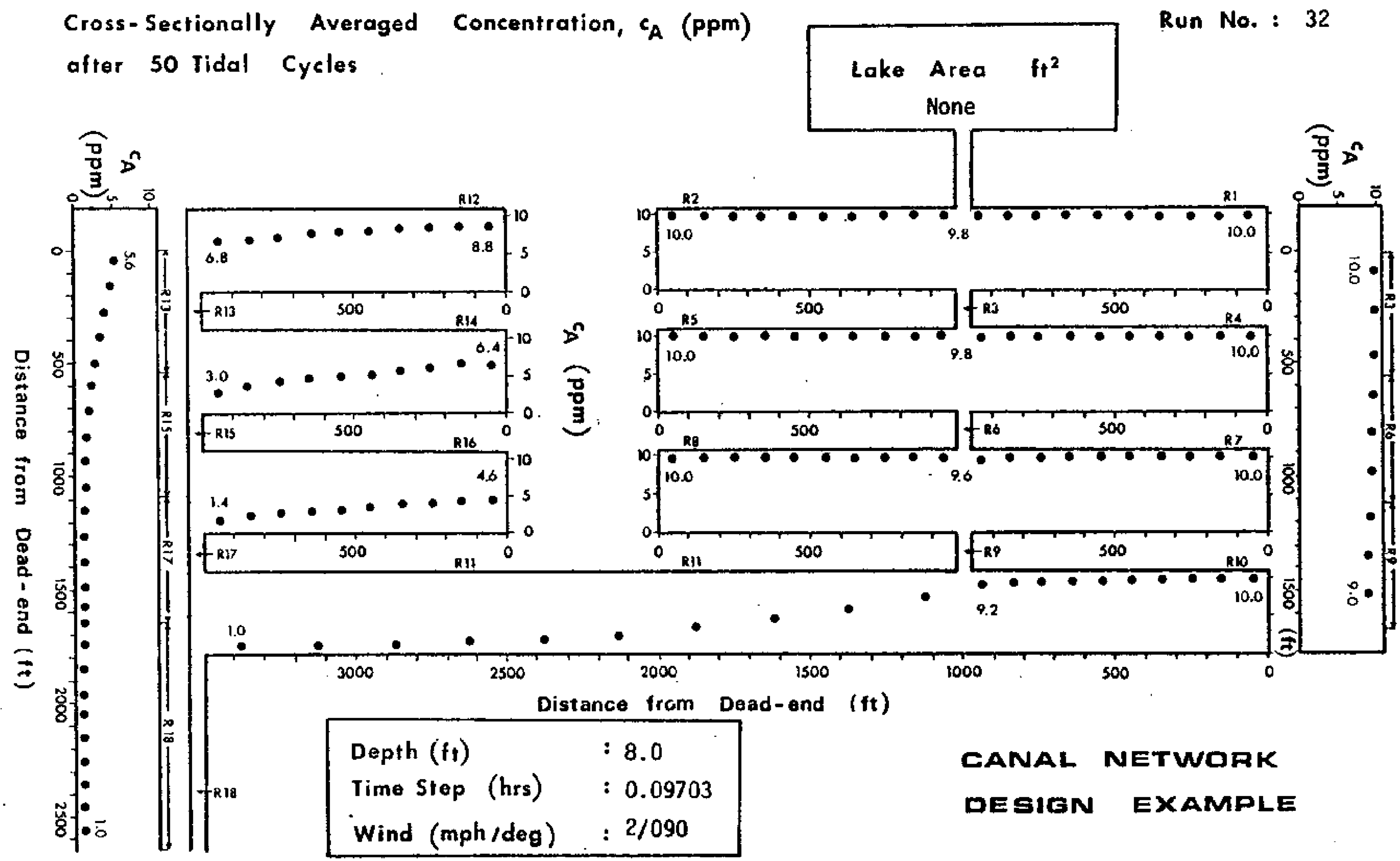


Figure 14.6 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With a Steady Wind of 2 mph from the East and No Lake (Results of Run No. 32).

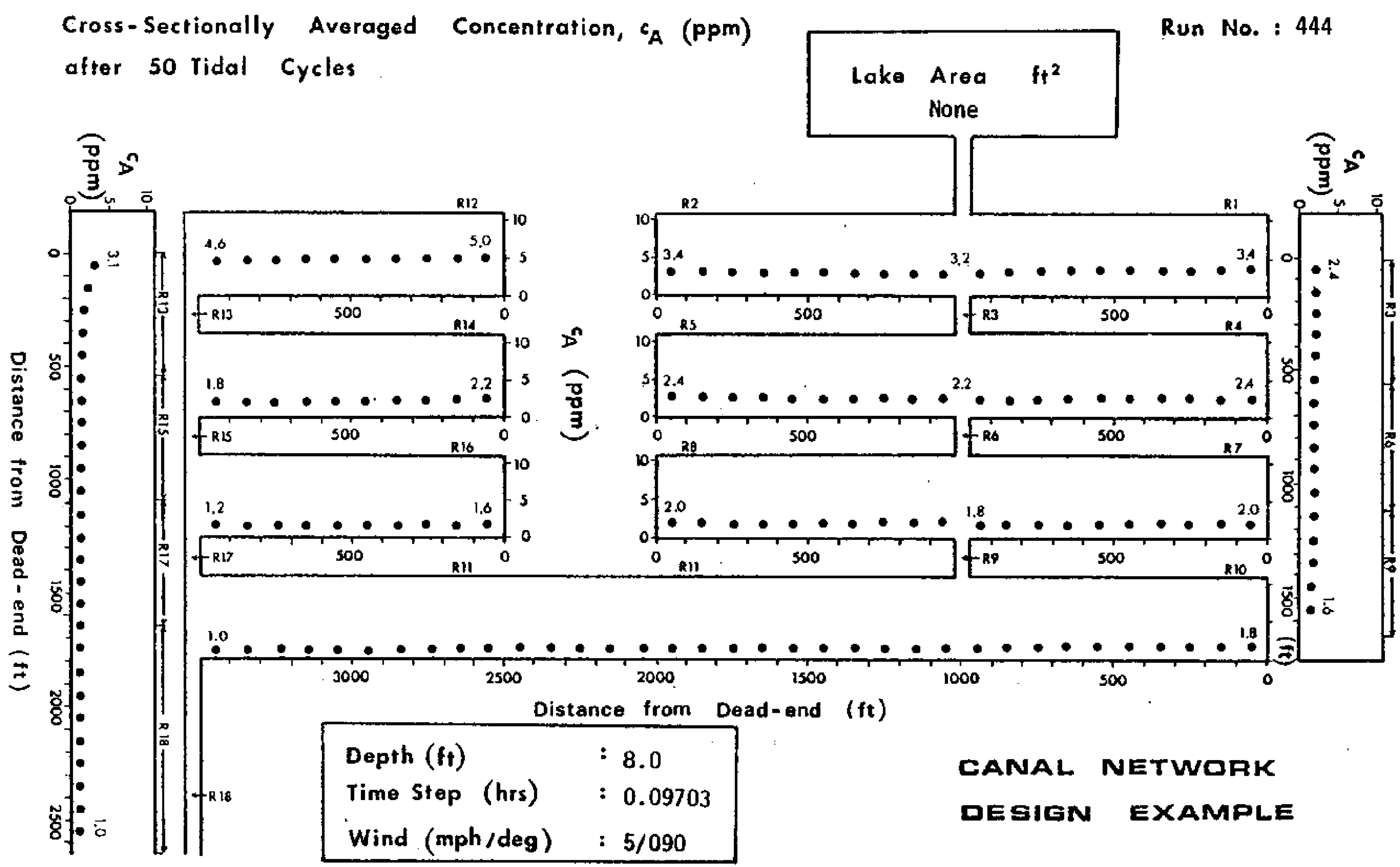


Figure 14.7 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With a Steady Wind of 5 mph from the East and No Lake (Results of Run No. 444).

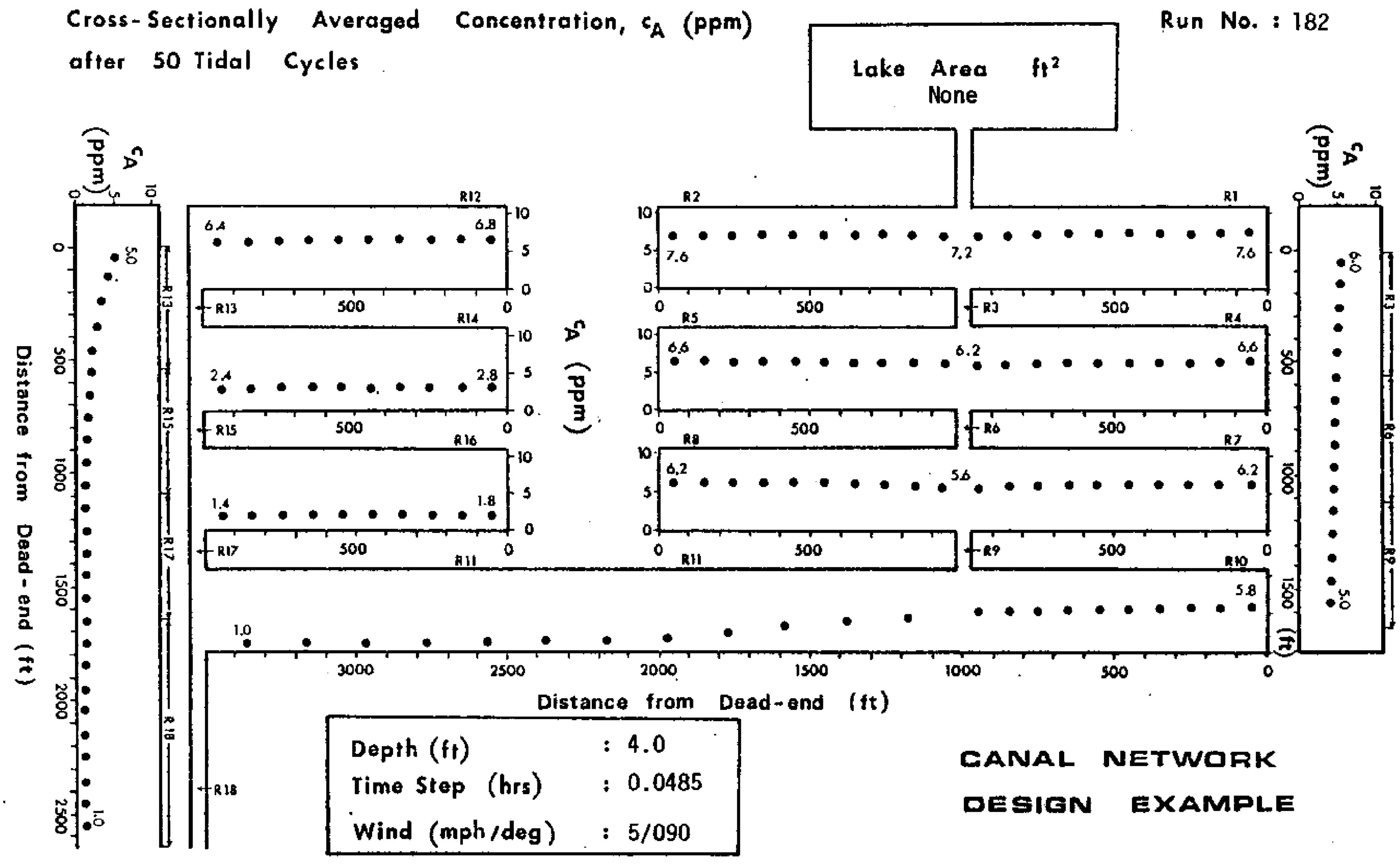


Figure 14.8 - Cross-sectionally Averaged Concentration Profiles in the 4-ft-deep Alternate Canal Network After Fifty Tidal Cycles, With a Steady Wind of 5 mph from the East and No Lake (Results of Run No. 182).

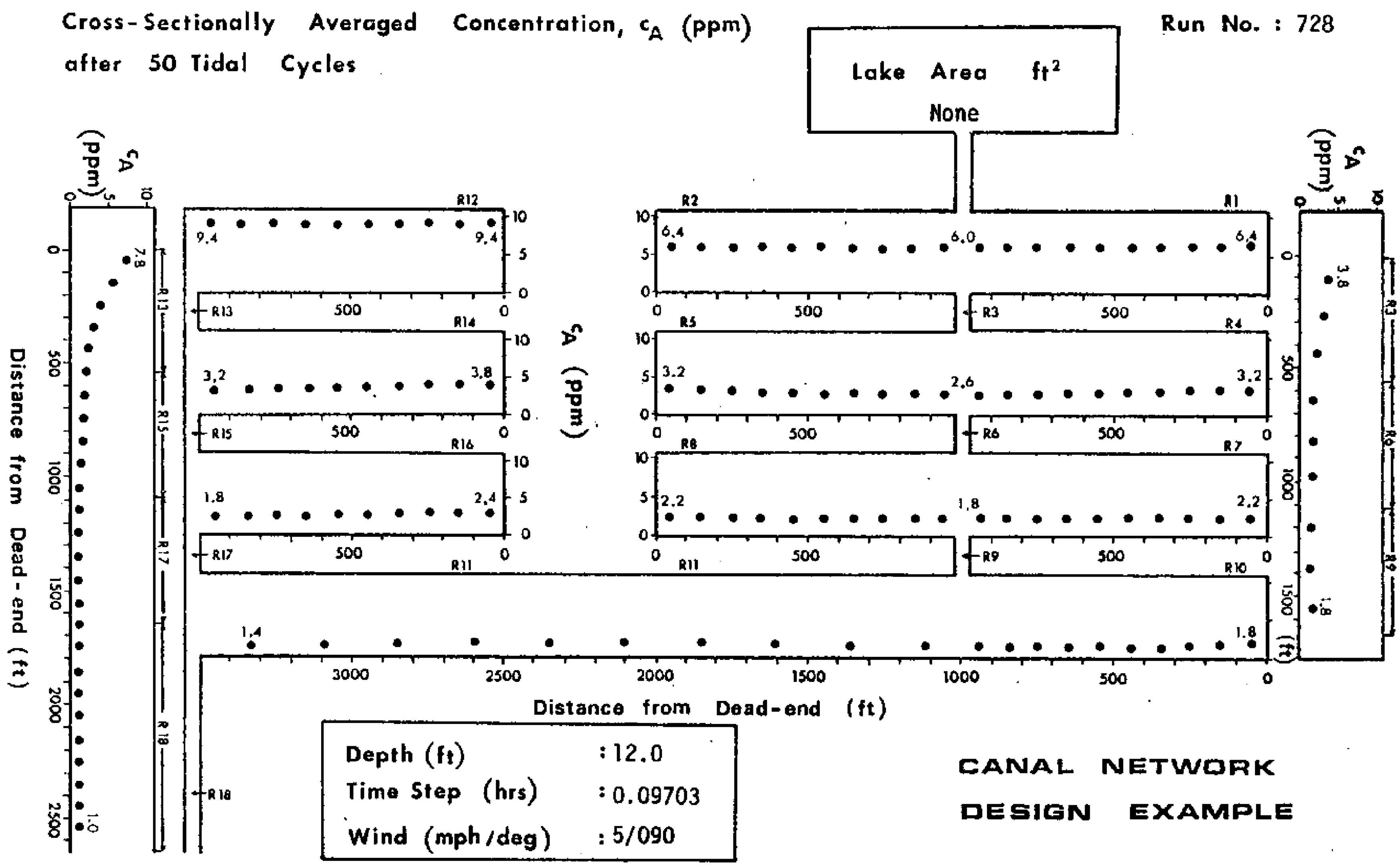


Figure 14.9 - Cross-sectionally Averaged Concentration Profiles in the 12-ft-deep Alternate Canal Network After Fifty Tidal Cycles, With a Steady Wind of 5 mph from the East and No Lake (Run No. 728).

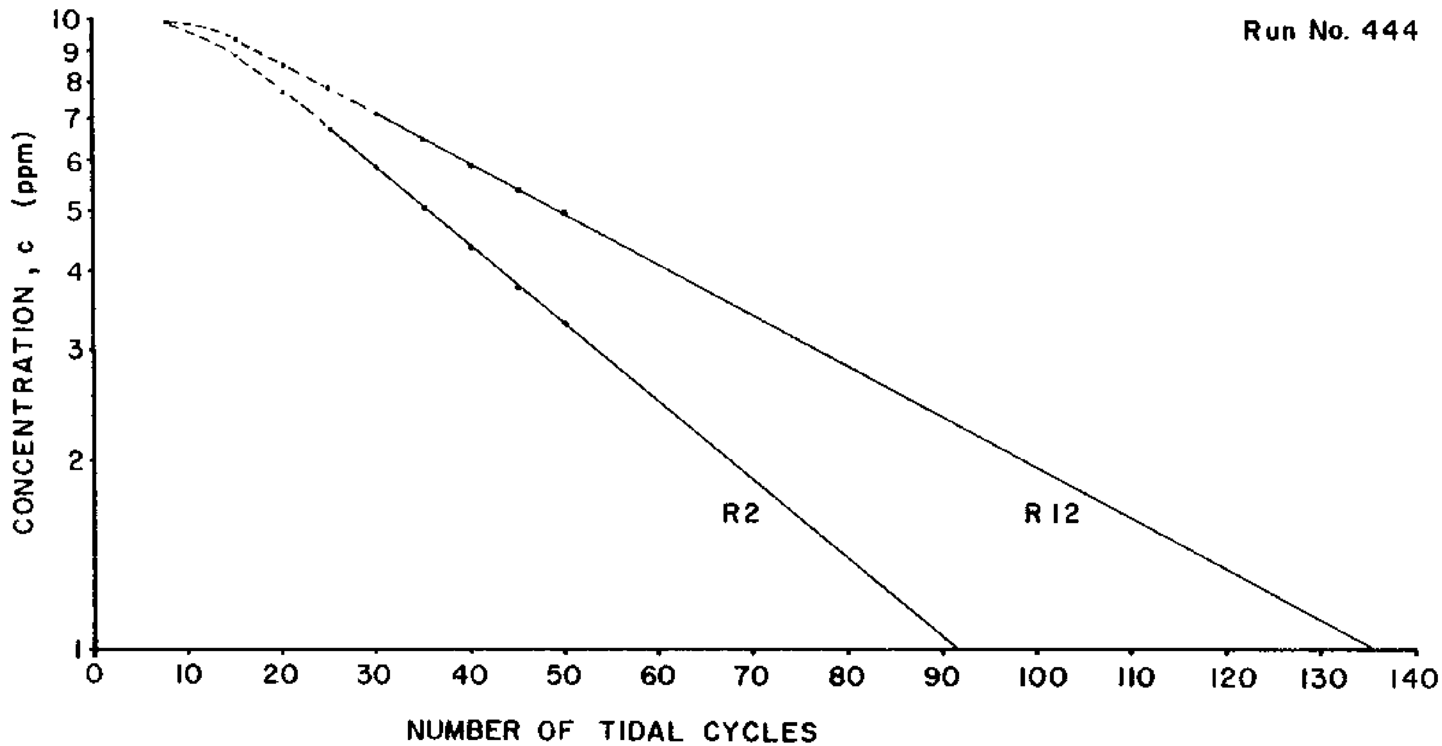


Figure 14.10 - Semilogarithmic Plots of Laterally Averaged Bottom Concentration at the Dead-ends of Canals R2 and R12 versus Number of Tidal Cycles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With a Steady Wind of 5 mph from the East and No Lake (Run No. 444).

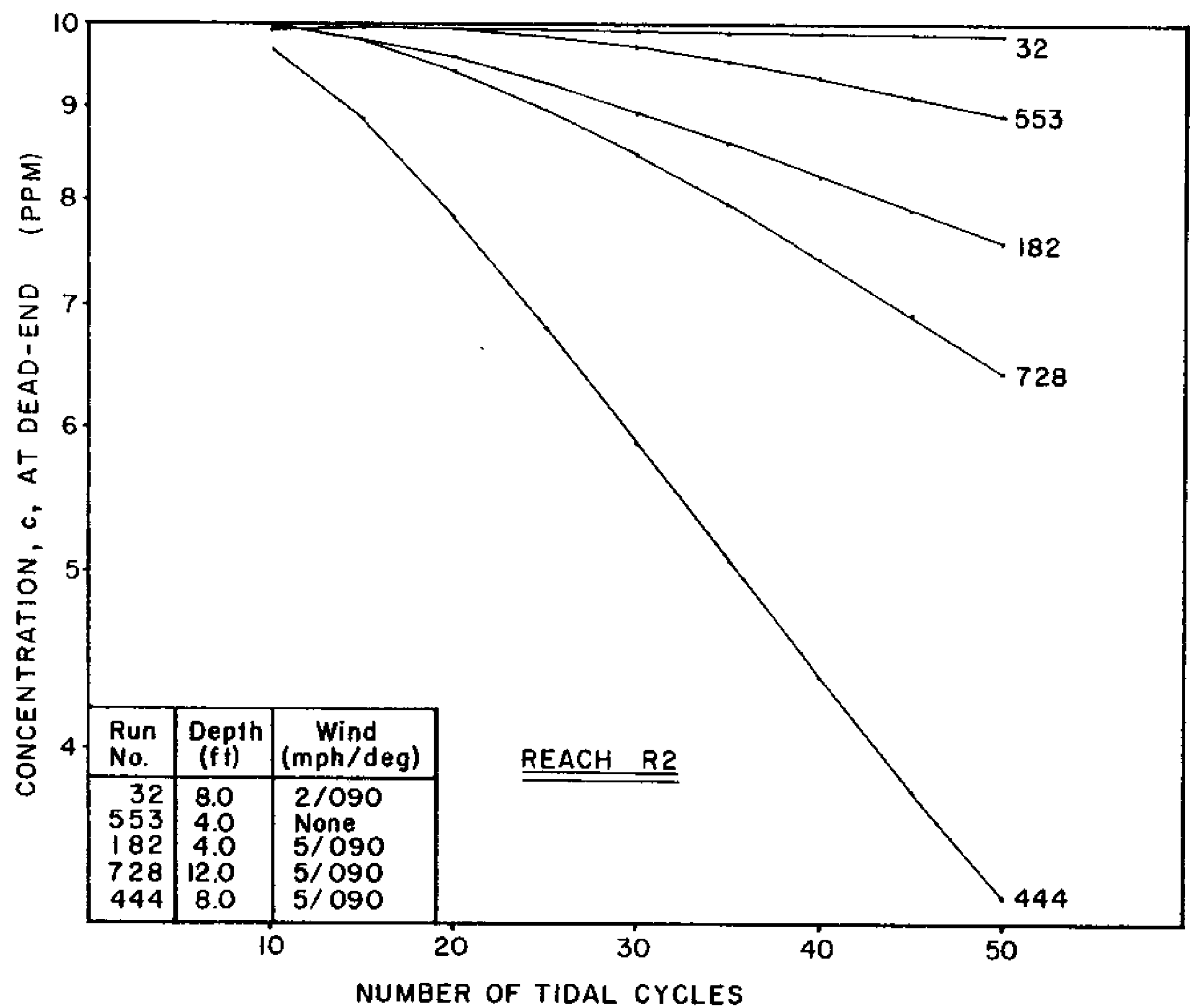


Figure 14.11 - Semilogarithmic Plots of Laterally-averaged Bottom Concentration at the Dead-ends of R2 Canals versus Number of Tidal Cycles for Various Combinations of Depths 4, 8 and 12 ft and East Winds With Velocities of 0, 2 and 5 mph (Results of Run Nos. 32, 182, 444, 553 and 728).

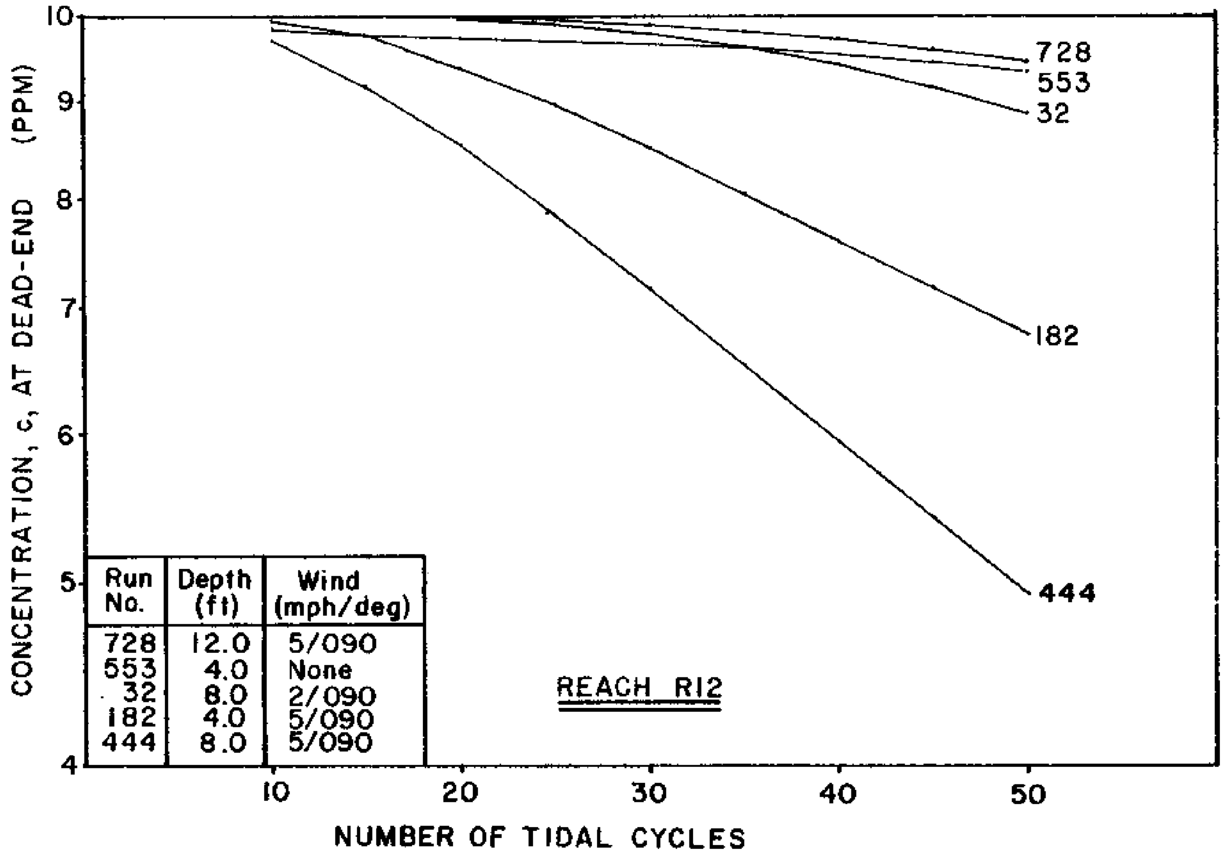


Figure 14.12 - Semilogarithmic Plots of Laterally-averaged Bottom Concentration at the Dead-ends of R12 Canals versus Number of Tidal Cycles for Various Combinations of Depths 4, 8 and 12 ft and East Winds With Velocities of 0, 2 and 5 mph (Results of Run Nos. 32, 182, 444, 553 and 728).

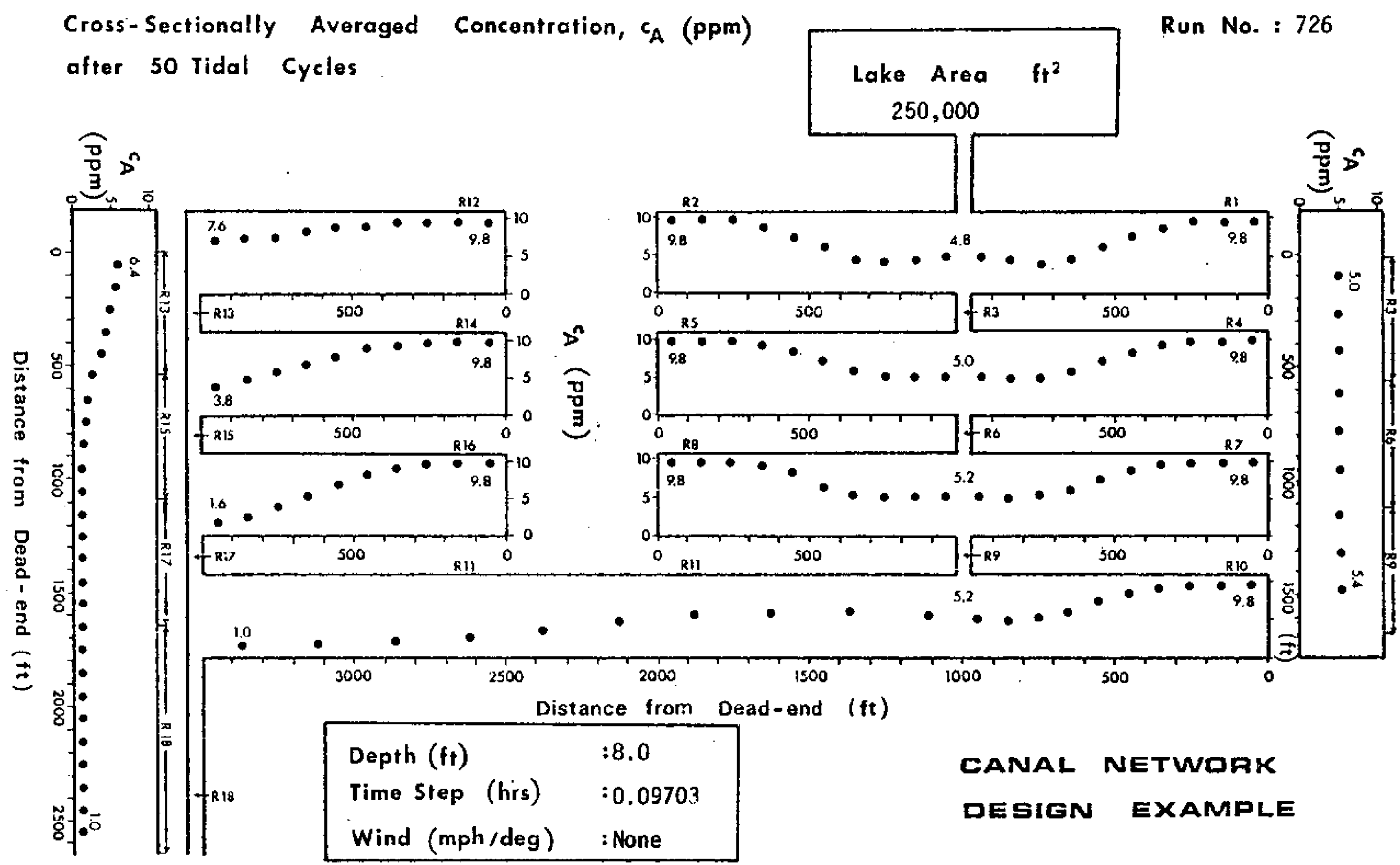


Figure 14.13 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With No Wind and a Lake With a Surface Area of 15 percent of Canal Network Surface Area (Run No. 726)

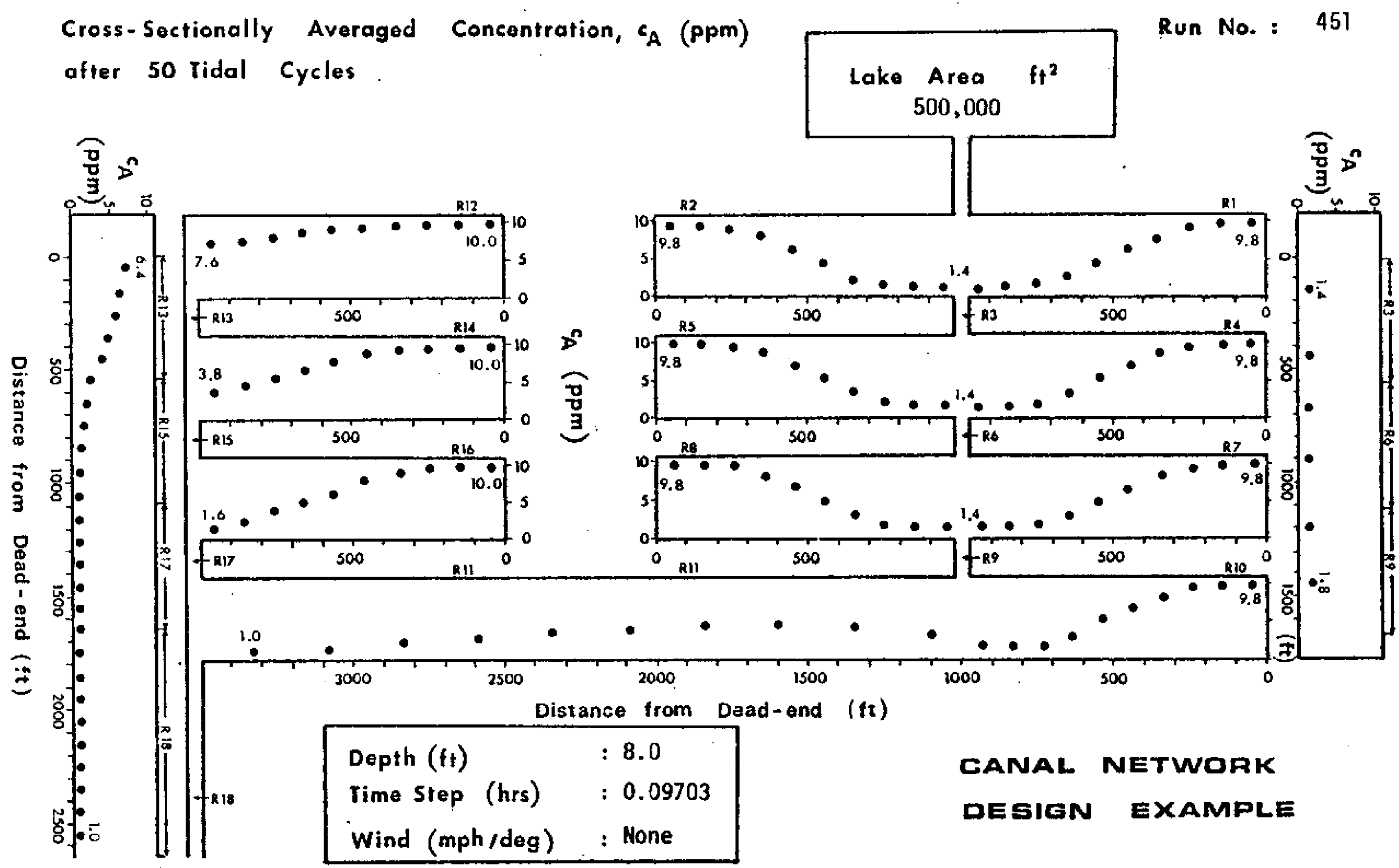


Figure 14.14 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With No Wind and a Lake With a Surface Area of 30 percent of Canal Network Surface Area (Run No. 451).

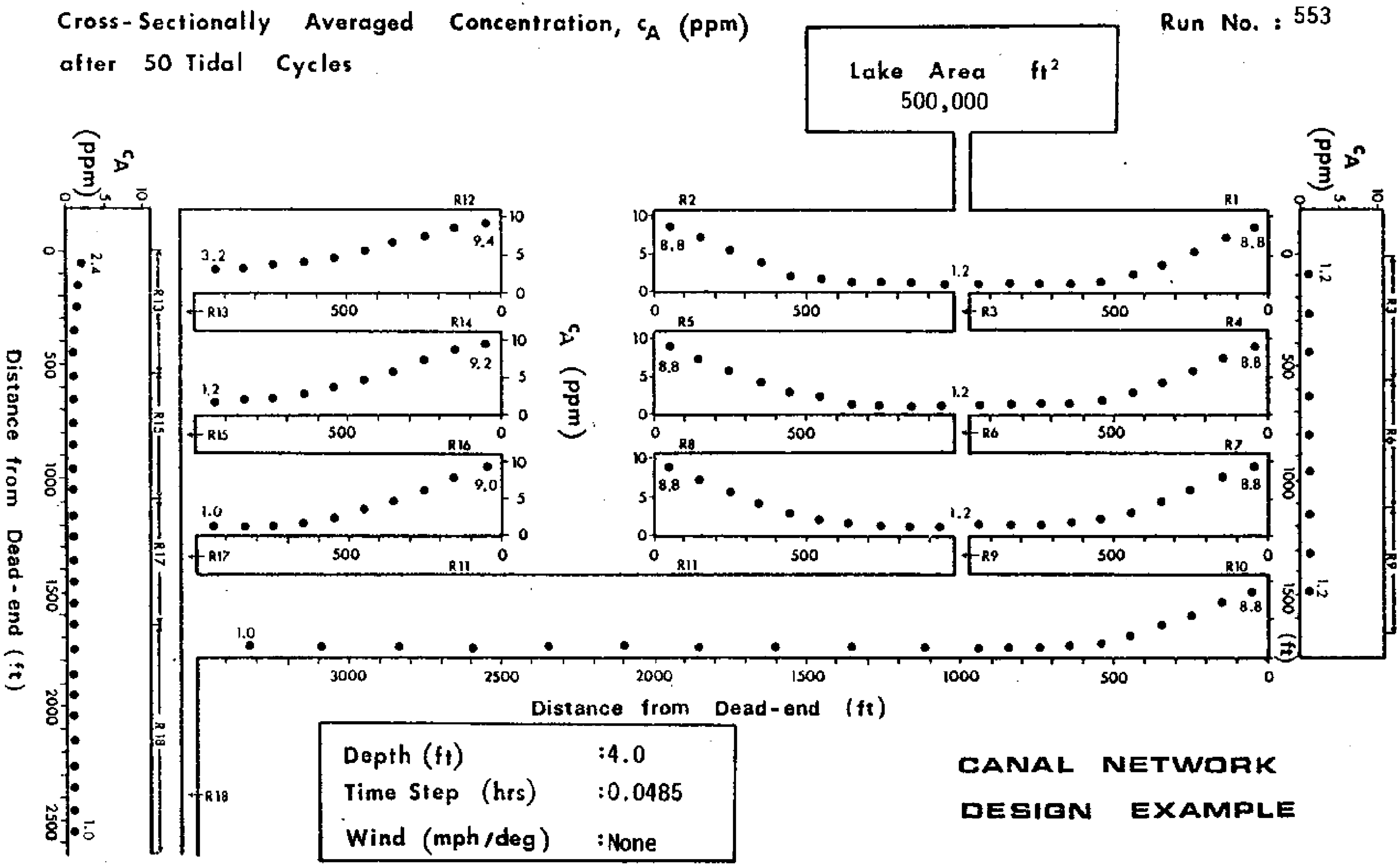


Figure 14.15 - Cross-sectionally Averaged Concentration Profiles in the 4-ft-deep Alternate Canal Network After Fifty Tidal Cycles, With No Wind and a Lake With a Surface Area of 30 percent of Canal Network Surface Area (Run No. 553).

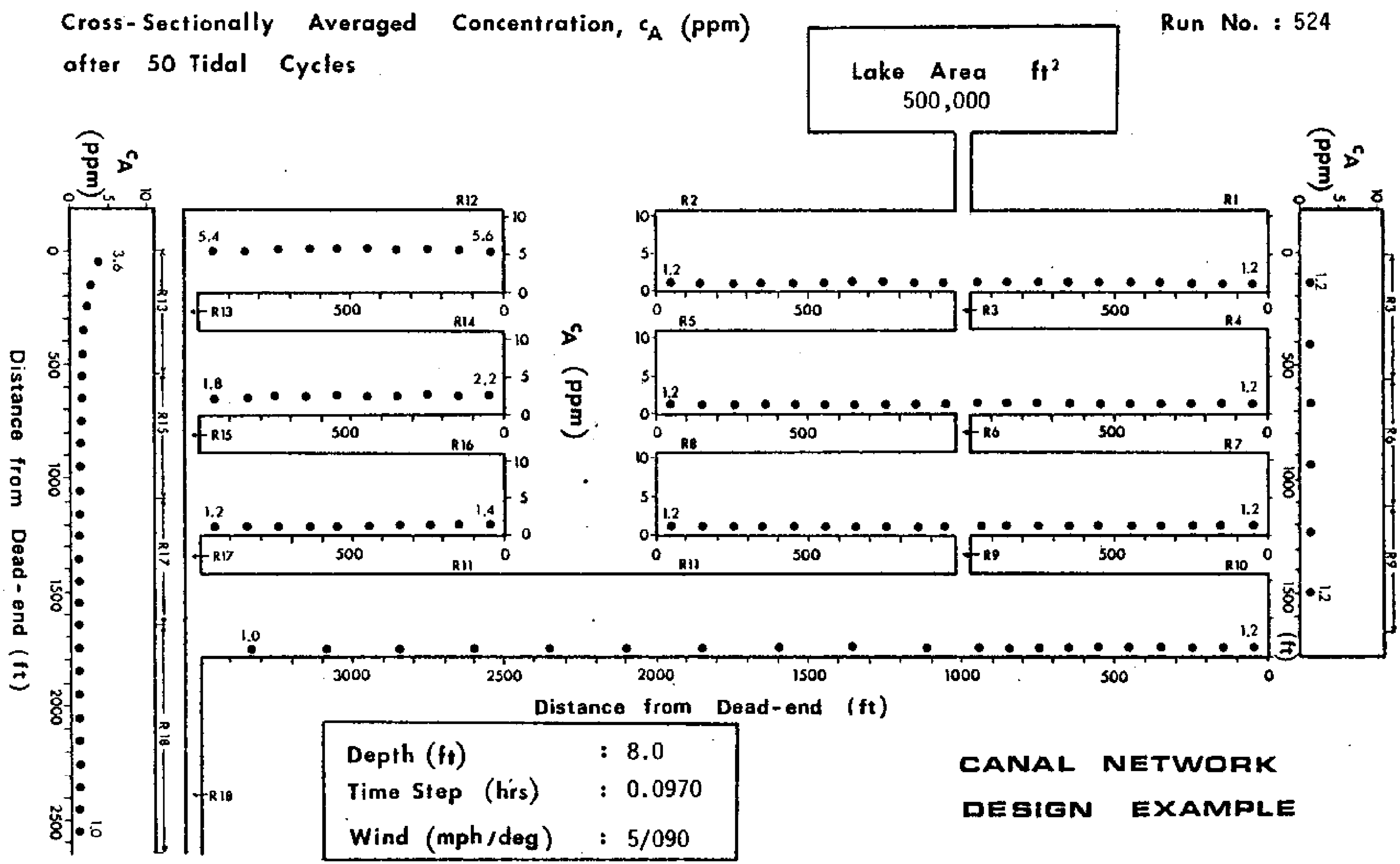


Figure 14.16 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With East Wind of 5 mph and a Lake With Surface Area of 30 percent of Canal Network Surface Area (Run No. 524).

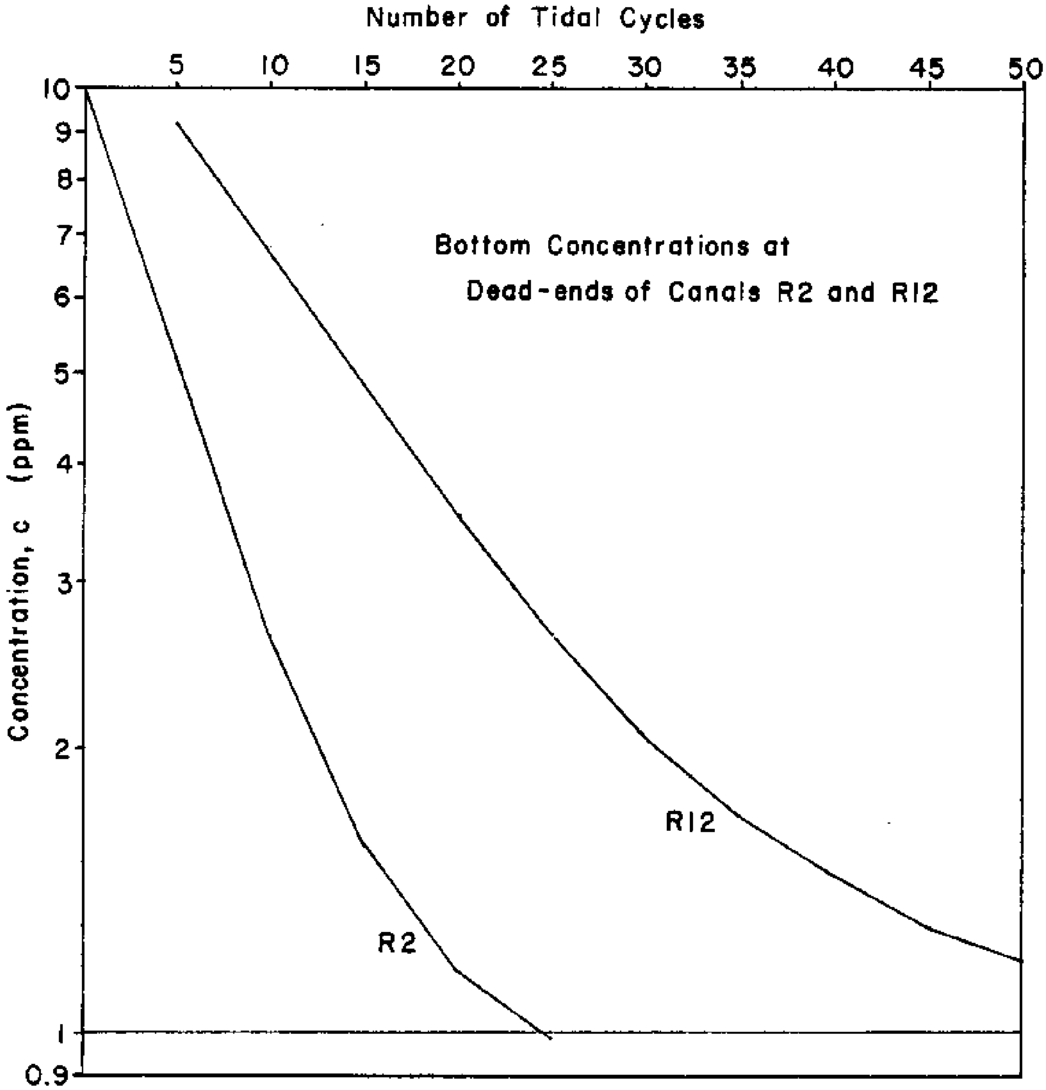
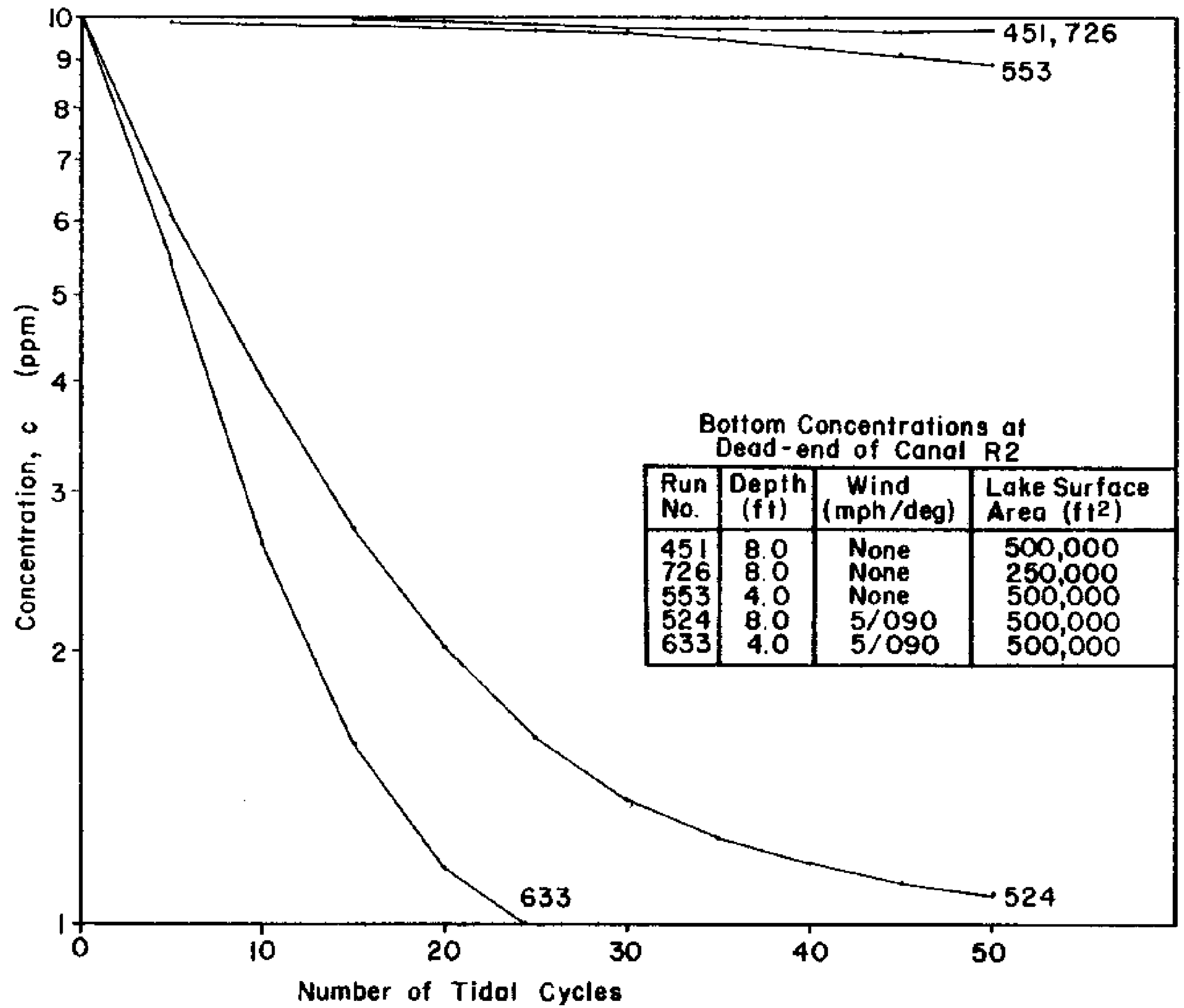


Figure 14.17 - Semilogarithmic Plot of Laterally-averaged Bottom Concentration at the Dead-ends of Canals R2 and R12 versus Number of Tidal Cycles for 4-ft-deep Alternate Canal Network After Thirty and Fifty Tidal Cycles, With Lake Surface Area of 30 percent of Network Surface Area and With a Steady Wind of 5 mph From the East (Run No. 633).



Run No.	Depth (ft)	Wind (mph/deg)	Lake Surface Area (ft^2)
451	8.0	None	500,000
726	8.0	None	250,000
553	4.0	None	500,000
524	8.0	5/090	500,000
633	4.0	5/090	500,000

Figure 14.18 - 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).

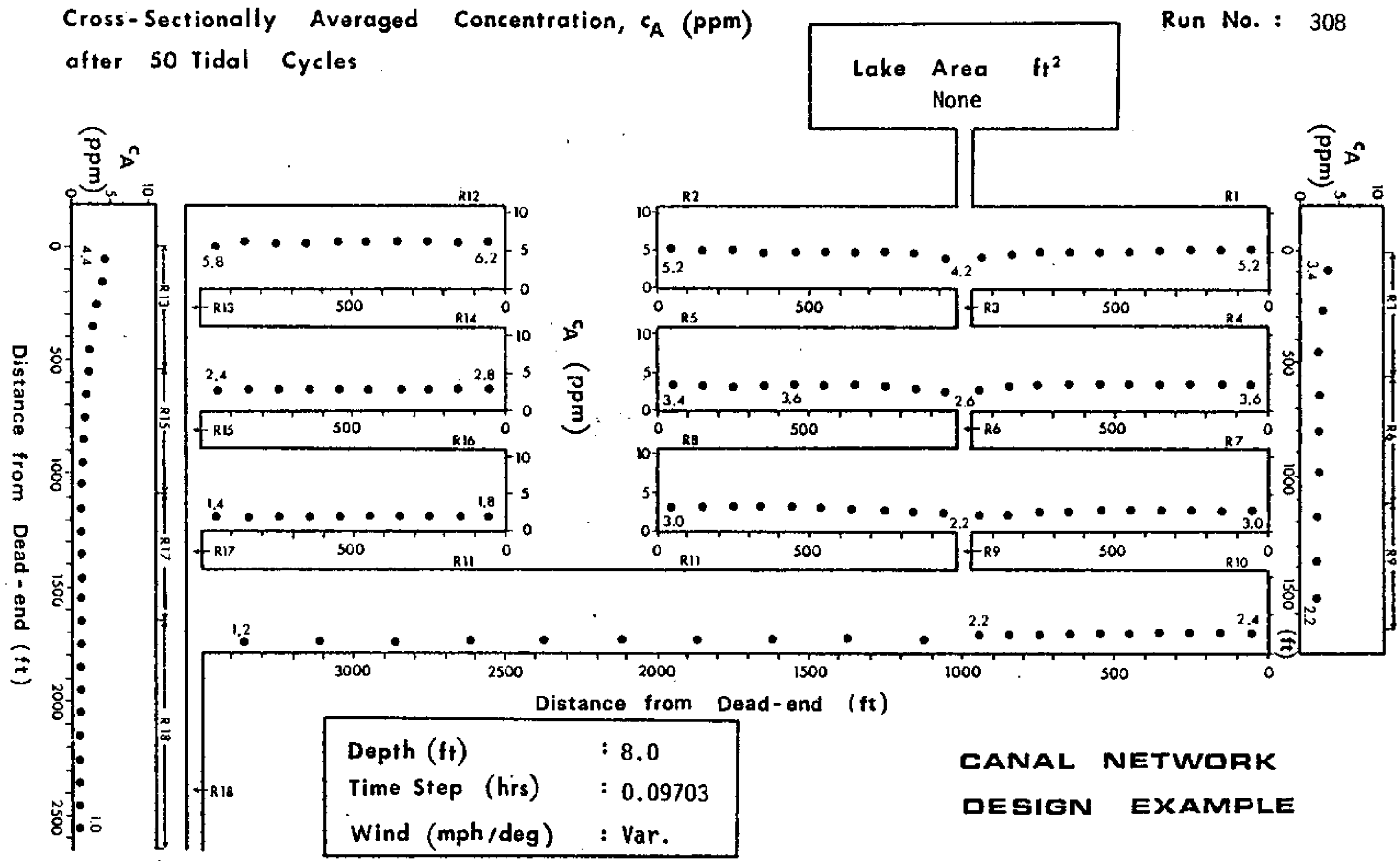


Figure 14.19 - Cross-sectionally Averaged Concentration Profiles in the 8-ft-deep Trial Canal Network After Fifty Tidal Cycles, With a Variable Wind Speed and Direction and No Lake (Results of Run No. 308).

Table 14.1 - 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 14.2 - Summary of Simulations of the Original, the Trial and Some Alternative Canal Network Designs.

Design	Mean Depth (ft)	Wind 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	14.3
Trial	8.0	none	none	no	no	443	14.4
	8.0	2/090	none	no	yes	32	14.6
	8.0	5/090	none	yes	yes	444	14.7
	8.0	none	250,000	no	no	726	14.13
	8.0	none	500,000	no	no	451	14.14
	8.0	5/090	500,000	yes	yes	524	14.16
	8.0	variable	none	no	no	308	14.19
Alternate	4.0	none	none	no	no	486	14.5
	4.0	5/090	none	yes	yes	182	14.8
	12.0	none	none	yes	yes	728	14.9
	4.0	none	500,000	yes	no	553	14.15
	4.0	5/090	500,000	yes	yes	633	14.17

CHAPTER 15

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS FOR CANAL DESIGN MANAGEMENT

15.1 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 developments.

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 interrelationships 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 separate 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 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 both the July and the October cases one value for the vertical momentum transfer coefficient, N_z, and one value for the background dispersion coefficient, E_o, provided the best fit of simulated concentration profiles in both cases. 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 form of the saltwater intrusion is likely to be 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 variability 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. This report summarizes some of the development considerations and legislative considerations involved and shows what kinds of overall design objectives should be developed to assure compatibility with both the developer's economic limitations and the legislative constraints that apply in the site's locality. The report then shows how 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 builders' 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 persons expert 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. This report outlines the types of information needed for planning and additional sources for relevant historical information and published data.

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. This report describes the requirements of such surveys in terms of data, instrumentation and support equipment, as well as the reduction and analysis of such data.

A canal network can be 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 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.

15.2 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 wetlands which has already been altered from its natural state, or one that has been created on dredge spoil and is only marginally useful. The areas which may be suitable, provided they do not naturally perform some other more important function, such as providing aquifer recharge, irreplaceable 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 in the report, 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. On the one hand 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 by 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. Descriptions of the type of instruments and support equipment which have been found successful for collecting the necessary data are included in this report.

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 needed 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 influenced by density gradients, 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 concentrations in these bottles should be measured daily.

5. The method of continuous dye concentration recording along the centerline of the canal should be modified to a sampling process at suitable cross-sections rather than a continuous measurement. At least one sample should be obtained 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.

Specific conclusions relating to the development of the model for the canal design project are included in Chapter 9. This model is a unique design tool which enables the canal designer to perform variability tests economically and with a variety of input conditions.

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 which will result in desirable hydrodynamic characteristics. Consideration of wind characteristics at the site is a *principal* concern at this stage of design. The following principles should be followed:

1. To ensure adequate flushing of the entire network, the *finger* canals should all 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 1.) above.
3. Space should be made available 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, near dead-ends the velocity is negligible and spreading of substances can only occur by diffusion.

2. The effect 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 throughout a canal network. In this case reaches with downstream winds flush the fastest, and reaches with upstream winds still flush more rapidly than in the case 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, 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 accommodated 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 canal 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.

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 tidal entrance and the lake, and a decreasing effect with distance into the finger canals.

15.3 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.

Several different kinds of simulations have been illustrated in Chapters 13 and 14. 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 generalized, cost functions could be developed for a design optimization procedure.

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