Simulation of the Effects of Proposed Tide Gates on Circulation, Flushing, and Water Quality in Residential Canals, Cape Coral, Florida

By Carl R. Goodwin

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CONVERSION FACTORS, VERTICAL DATUM, AND ABBREVIATIONS

Multiply	By	To obtain
inch (in.)	25.4	millimeter
foot (ft)	0.3048	meter
foot per second (ft/s)	0.3048	meter per second
mile (mi)	1.609	kilometer
square foot per second	0.09290	square meter per second
square mile (mi^2)	2.590	square kilometer
cubic foot (ft^3)	0.02832	cubic meter cubic
foot per second	0.02832	cubic meter per second
acre	0.4047	hectare

Sea level: In this report, "sea level" refers to the Naional Vertical Datum of 1929 (NGVD of 1929)--a geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada, formerly called Sea Level Datum of 1929.

ADDITIONAL ABBREVIATIONS

Branched Lagrangian Model (BLTM)
degrees Celsius (°C)
milligrams per liter (mg/L)
milisiemens per centimeter (mS/cm) parts per thousand (ppt)

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Abstract

Decades of dredging and filling of Florida's low-lying coastal wetlands have produced thousands of miles of residential tidal canals and adjacent waterfront property. Typically, these canals are poorly flushed, and, over time, accumulated organic-rich bottom materials contribute to an increasingly severe degraded water quality. One-dimensional hydrodynamic and constituent-transport models were applied to two dead-end canal systems to determine the effects of canal system interconnection using tide gates on water circulation and constituent flushing. The model simulates existing and possible future circulation and flushing conditions in about 29 miles of the approximately 130 miles of tidally influenced canals in Cape Coral, located on the central west coast of peninsular Florida.

Model results indicate that tidal water-level differences between the two canal systems can be converted to kinetic energy, in the form of increased water circulation, by the use of one-way tide gate interconnections. Computations show that construction of from one to four tide gates will cause replacement of a volume of water equivalent to the total volume of canals in both systems in 15 to 9 days, respectively. Because some canals flush faster than others, 47 and 21 percent of the original canal water will remain in both systems 50 days after start of operation of one and four tide gates, respectively. Some of the effects that such increased flushing are expected to have include reduced density stratification and associated dissolved-oxygen depletion in canal bottom waters, increased localized reaeration, and more efficient discharge of stormwater runoff entering the canals.

INTRODUCTION

The increased demand for and limited availability of waterfront property suitable for residential development have combined to intensify pressures for dredging and filling many of the low-lying coastal wetlands of Florida. Prior to the early 1970's, dredging of tidal canal networks into wetlands and filling of adjacent marsh and uplands was a common practice. The resulting canals provided additional waterfront property, boat access to open water, fill material with which to cover natural vegetation and raise land to elevations suitable for building, and increased drainage efficiency (U.S. Environmental Protection Agency, 1975, p. 13-14).

Decades of coastal dredging and filling activities have resulted in the creation of thousands of miles of coastal residential canal systems in place of hundreds, and probably thousands, of square miles of highly productive wetlands habitat. These canal systems have repeatedly been shown to cause the development of undesirable water-quality conditions (Taylor and Saloman, 1968; Barada and Partington, 1972; U.S. Environmental Protection Agency, 1975; 1976a; 1976b) and, in some cases, to cause the accumulation of toxic material and pathogenic organisms in fine bottom sediments (Barada and Partington, 1972). The most common water-quality problems have been relatively low dissolved-oxygen concentrations in water at the bottom of the canals and, in older canal systems, elevated nitrogen concentrations. Significant numbers of pathogenic organisms also have been detected in bottom sediments such as those responsible for streptococcal, staphylococcal, stomach, ear, and urinary tract infections (Barada and Partington, 1972).

Many residential canals lack sufficient water circulation to disperse dissolved or suspended materials such as nutrients and fine sediment that may be transported to the canals by runoff from adjacent land areas. These conditions potentially can lead to anaerobic conditions on canal bottoms, the release of hydrogen sulfide gas, and the exchange of nutrients and heavy metals from bottom sediments to the water column (Southwest Florida Regional Planning Council, 1984). The relatively stagnant conditions also encourage the accumulation and overproduction of phytoplankton and algae, which grow within the canals in response to favorable energy and nutrient conditions. A number of studies have identified this lack of flushing as one of the major contributors to degraded conditions in coastal tidal canal systems (Barada and Partington, 1972; Morris, 1979; Southwest Florida Regional Planning Council, 1984).

One of the most extensive examples of residential canal construction can be found in the city of Cape Coral in western

Lee County (fig. 1). It has been suggested that increased circulation and flushing might be achieved in the tidal canals of Cape Coral and other locations by selective placement of canal interconnections and tide gates to take advantage of tidal stage differences within the system. The potential energy available in the stage differences induced by the tidal characteristics of the system could be converted to kinetic energy sufficient to markedly increase water velocities and circulation.

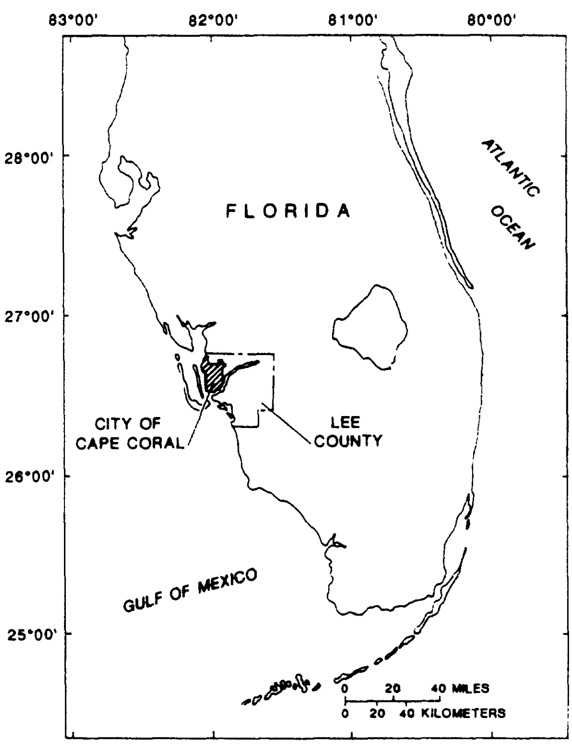


Figure 1. Location of Cape Coral.

Between April 1985 and September 1987, a study to determine the potential for improving water circulation and constituent flushing by installing tide gates in part of the canal system in Cape Coral was undertaken by the U.S. Geological Survey in cooperation with the Florida Department of Environmental Regulation. The objectives of this study were to:

1. Determine the probable increase in circulation and flushing that can be induced by construction of tide gates in a part of the canal system of Cape Coral, and
2. Evaluate the probable water-quality effects on the canals and the adjacent Caloosahatchee River that could result from increased canal circulation and flushing.

Purpose and Scope

This report presents the methods, results, and conclusions of the aforementioned study. Data-collection activities included the determination of the physical, chemical, tidal, and dispersive characteristics of the canal system. Numerical modeling techniques were applied to simulate tidal flow, circulation, and flushing dynamics in the present canal system and many possible variations in the present system that could result from the construction of tide gates and canal interconnections. Model results and measured water-quality conditions were then used to evaluate some probable effects of tide-gate installation on water quality in the canals and Caloosahatchee River.

Previous Studies

Some documentation of water-quality conditions in the tidally affected, saltwater parts of the Cape Coral canal system is available from studies by the U.S. Environmental Protection Agency (1976a) and the Southwest Florida Regional Planning Council (1984). In 1986, a program of periodic water-quality data collection in the freshwater and the saltwater canal systems was initiated by the city of Cape Coral (Douglas Morrison, City of Cape Coral, written commun., 1987). These studies generally indicate that the water-quality problems existing in parts of the Cape Coral canal systems are similar to, but not yet as severe as, conditions in dead-end canals in other parts of Florida that have inadequate flushing, that receive stormwater runoff, and that accumulate fine organic sediments and associated contaminants.

Improved flushing of dead-end canals has long been recognized as an important part of solving the problem of dead-end canal eutrophication (Morris and others, 1978; Morris, 1979). State permitting regulations now require a detailed analysis of flushing prior to the construction of new canals. One method used to improve flushing and help control tidewater intrusion into stormwater runoff pipes during high tides has been the use of a flapper or tide gate. The gate is hinged to close when the tidal stage becomes greater than the upstream freshwater head. The Florida Department of Transportation has extended this concept and used it to improve flushing in an isolated tidal area of Tampa Bay where the satisfactory operation of a 40-ft-wide tide gate was reported by Morgan and others (1984). Identification of the best practice to use to improve flushing and improve water quality in existing dead-end canal systems, however, is still a matter of debate.

Acknowledgments

The encouragement of this study by the staff in the Fort Myers office of the Florida Department of Environmental Regulation and the willing assistance of Douglas Morrison, City of Cape Coral, in supplying water-quality and other information is gratefully acknowledged. David H. Schoellhamer, research hydrologist with the U.S. Geological

Survey, provided assistance in modifying the BRANCH model to incorporate tide-gate functions.

APPROACH

On the basis of tidal stage data from a series of temporary gages established throughout the Cape Coral canal system, an appropriate part of the tidal canal system was chosen as the study area. Data from the temporary gages also were used, in conjunction with available information on the physical dimensions of the canals, to develop and test early versions of both the hydrodynamic and constituent-transport models used during the study.

Computed information from the preliminary models was then used in the design of a data-collection program that would define the attributes of the study area to assure reliable and verifiable study results. Data collection included:

1. Longitudinal definition of water depths in one major part of the study area using boat-mounted fathometry equipment to characterize the present physical dimensions of the system,
2. Continuous measurement of tidal stage at seven sites during the study period to provide information to drive the hydrodynamic model and to help evaluate the degree of similitude between the hydrodynamic model and the real canal system,
3. Repetitive measurement of tidal discharge for a period of 24 hours at five sites to provide a second and more sensitive evaluation of hydrodynamic model similitude,
4. Continual measurement of dye tracer concentrations that resulted from simultaneous slug injections in three parts of the study area for a period of 40 hours at seven sites to help evaluate the degree of similitude between the constituent-transport model and the real system,
5. Longitudinal definition of dye concentrations for a period of 40 hours after dye injection by using boat-mounted fluorometry equipment to provide additional data for evaluation of the constituent-transport model,
6. Measurements of nutrient and dissolved-oxygen-concentrations and selected water-quality characteristics (temperature, specific conductance, and pH) at about 6-hour intervals to define summertime water-quality conditions during the 40-hour dye dispersion period,
7. A one-time measurement of nutrient and dissolved-oxygen concentrations and selected water-quality characteristics throughout the study area and in the Caloosahatchee River to define water-quality conditions during the winter, and
8. Twelve measurements of nutrient and dissolved-oxygen concentrations and selected water-quality characteristics at each of nine sites during a 17-month period to define their seasonal variability.

The calibrated and verified hydrodynamic model was applied to determine the optimal placement of from one to four tide gates in the study area. A volume-weighted average of tide-induced circulation was used as the optimizing parameter because several lakes within the canal system are known to have more extensive and frequent degraded water quality than do the canals.

The constituent-transport model was developed and applied to the existing condition and the four optimal tide-gate placement options to compare computed rates of constituent flushing in the study area. This information was then used in conjunction with measured water-quality data to evaluate the probable effects that tide gates might have on some water-quality characteristics of the canals and the Caloosahatchee River.

DESCRIPTION OF STUDY AREA

The city of Cape Coral is at the mouth of the Caloosahatchee River (fig. 2), which forms the southern and southeastern boundaries of the city. The western boundary of the city consists of a saltwater and mangrove marsh that separates the city from Matlacha Pass, a body of water designated as an aquatic preserve by the State of Florida. The Caloosahatchee River drains a basin of about 1,380 mi^2 in southwest Florida (Hammett, 1988, p. 17). River discharges during the study period are shown in figure 3 along with rainfall at Fort Myers.

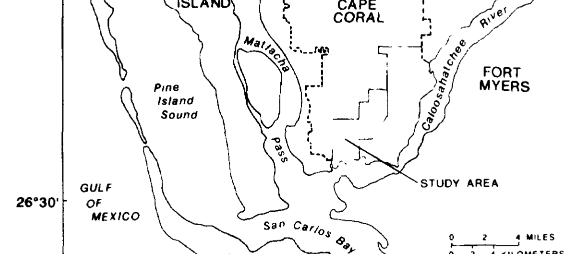


Figure 2. Study area within region of tidally affected saltwater canals in Cape Coral.

Information from the Southwest Florida Regional Planning Council (1984) indicates that the 104-mi^2 area of the city of Cape Coral contains about 400 mi of saltwater and freshwater residential canals that were dredged into native wetlands and uplands. Around this canal system, there are approximately 138,000 platted single-family homesites and

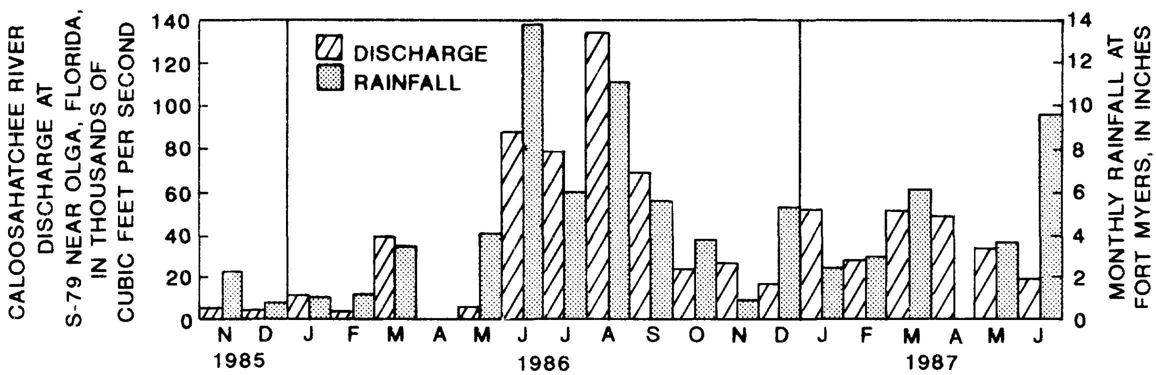


Figure 3. Caloosahatchee River streamflow and Fort Myers rainfall.

additional multifamily, commercial, industrial, and community areas that are being developed at a rapid pace. The city's permanent population in 1980 was measured at 32,000, with a total population potential of about 400,000 (Southwest Florida Regional Planning Council, 1984). With the fastest percentage growth rate of any city in the country in 1984 and the probable associated increases of a wide range of environmental contaminants (including nutrients, trace metals, fine sediment, pesticides, and other synthetic organic compounds), there is great potential for water-quality degradation and eutrophication in the canals and lakes of the city (Leopold, 1968).

Of the 400 mi of canals that honeycomb the entire city, about 130 mi along and near the Caloosahatchee River are tidally affected and contain saltwater. The freshwater canals are separated from the saltwater canals by a series of weir structures whose crests are several feet above sea level. The saltwater section is shown in figure 4 and is subdivided into four functional units for purposes of this study using a naming convention modified from that of Connell, Metcalf, and Eddy, Inc. (1979).

The River canal system is composed of canals whose farthest inland extent is generally less than a mile from the Caloosahatchee River. This system consists of a number of small canal systems that open directly to the river and are not a part of this study. The Rubicon canal system generally parallels and extends about a mile farther inland than the River system, has three tidal openings to the Caloosahatchee River, contains about 19.5 mi of canals with no lakes, and has one source of freshwater from the freshwater canal system. The Bluejay canal system extends inland more than 2 mi from its one opening to the Caloosahatchee River, contains about 16.0 mi of canals, 0.31 mi^2 of lakes, and has no source of freshwater input other than direct runoff from land adjacent to the canals within the system itself. The San Carlos canal system extends inland about 2.5 mi from its inland boundary with the River canal system, has only one tidal opening, contains about 12.8 mi of canals, 0.06 mi^2 of lakes, and has two sources of freshwater input from the freshwater canal system (fig. 4) in addition to locally derived runoff.

The Bluejay canal system was initially chosen as the study area for this investigation because it is a self-contained unit with only one tidal opening and no outside sources of freshwater. The system is fully developed and the structure of primary, secondary, and tertiary canals is well defined. It also has several large lakes that may have unique effects on both the hydrodynamics and constituent transport within the system. From a pragmatic viewpoint, the Bluejay canal system has less residential development than the other canal systems, making more sites available for possible future tide gates.

After preliminary model simulations of the Bluejay canal system, the primary study area was expanded to include the San Carlos canal system in order to assure realistic results. Otherwise, the hydraulic interaction between the two systems that would result from tide gate installation could not have been adequately approximated. The area selected for detailed study, including these two canal systems, is outlined in figure 4. Most of the data collection was confined to this enlarged study area, with the exception of one tide gage operated in the Rubicon canal system.

Bluejay and San Carlos Residential Canal Systems

Canal and Lake Dimensions

The accurate simulation of water and constituent motion in the canals of Cape Coral requires a knowledge of the physical dimensions of the various canals and lakes, including canal constrictions such as at bridges. During this study, a combination of new measurements and available data were used to establish these dimensions.

The length, depth, top width, and cross-sectional shape of many canals in the Bluejay and the San Carlos canal systems have been compiled by Connell, Metcalf, and Eddy, Inc. (1979), and are summarized here. Canal-top widths in

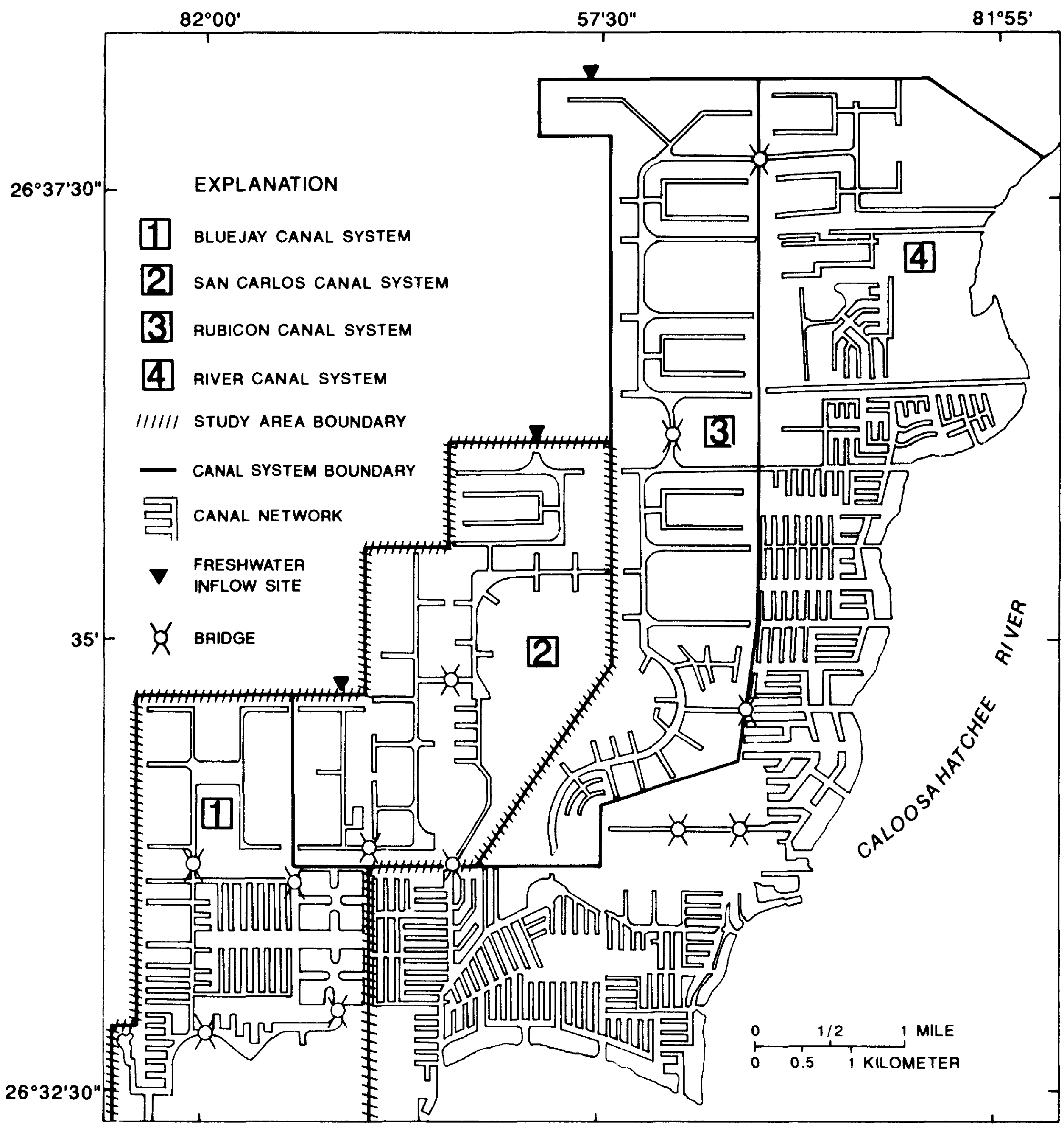


Figure 4. Study area and four functional units of saltwater canals.

the Bluejay canal system are generally 200 ft in the areas closest to the Caloosahatchee River, although most canals are 100 ft wide. In the San Carlos canal system, canal-top widths are uniformly 120 ft in the eastern part of this system and 100 ft in the western part. Exceptions to these dimensions occur at bridges and other constrictions and at several significantly wider areas that are referred to as lakes (fig. 5).

To determine if the available depth information would be adequate to use in this study, a fathometer survey was performed along the centerline of many canals in the Bluejay system. The path of the survey is given in figure 5 and numbered locations are keyed to longitudinal depth profiles that are shown in figure 6. Depth profiles from Connell, Metcalf, and Eddy, Inc. (1979), are also shown where they are available. Comparisons show that depth differences are often substantial, particularly in canals near the Caloosahatchee River, and that measured depths are generally greater than those given in the literature.

Measured canal depths in the Bluejay system, relative to sea level, range from about 30 ft near the mouth, where large quantities of material were dredged, to only 1 or 2 ft near some of the canal dead ends. Several canals close to the Caloosahatchee River have depths typically greater than 15 ft except near bridges where depths rapidly decrease to 6 or 8 ft. In some instances, rapid depth changes are not associated with bridges.

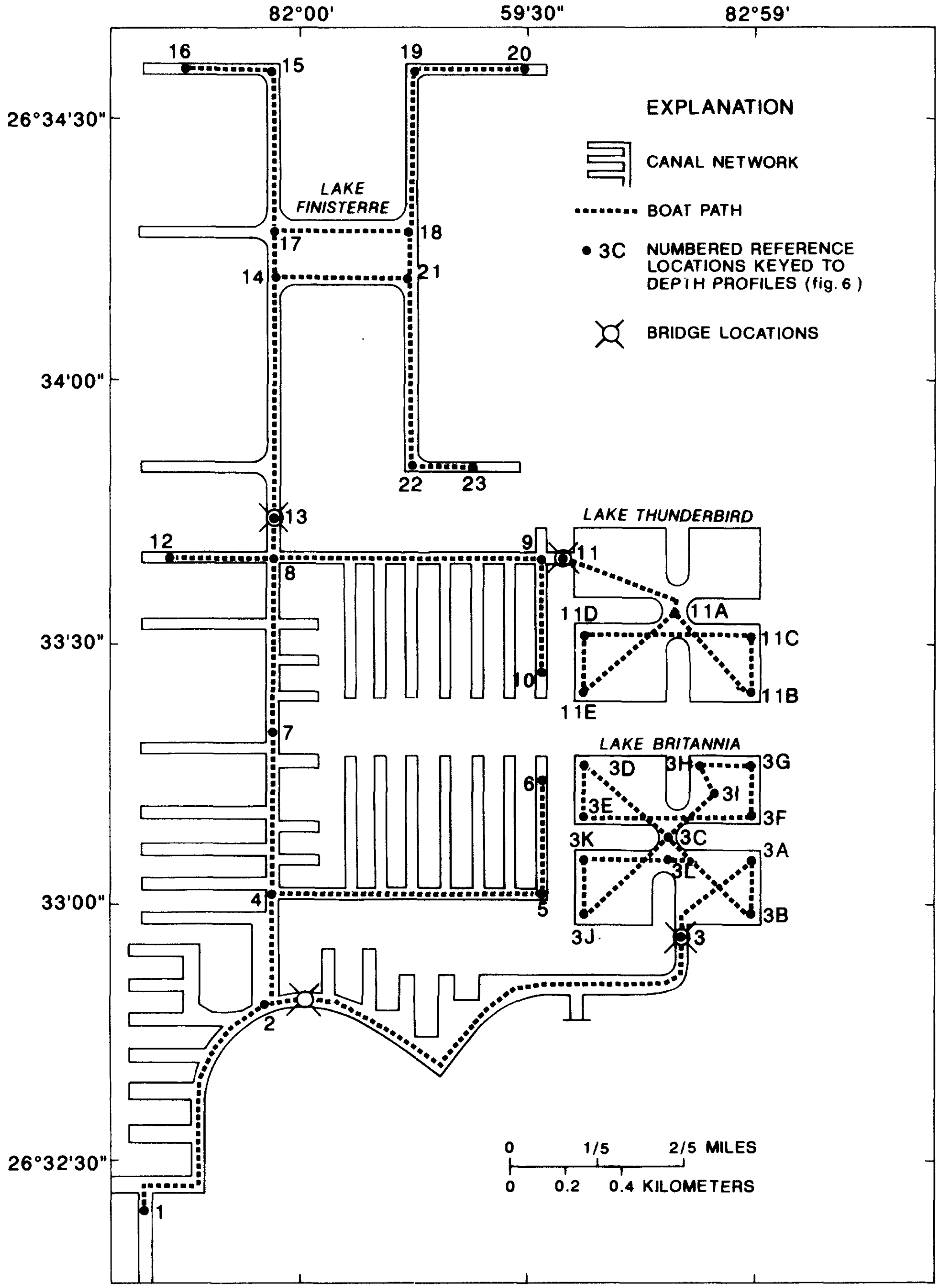
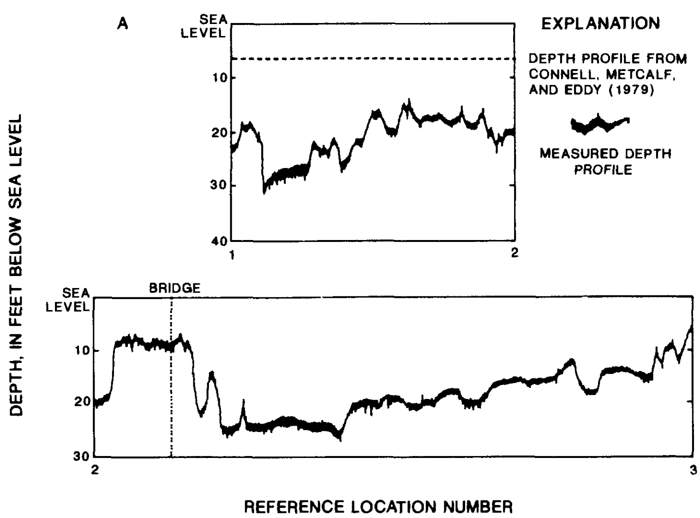
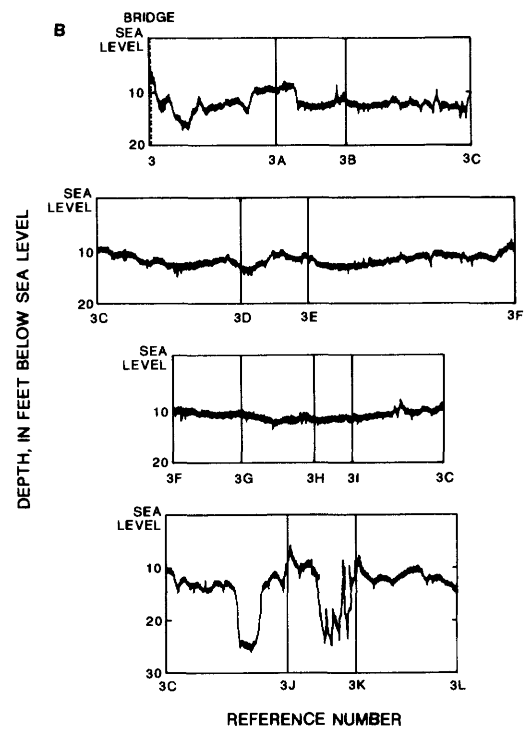


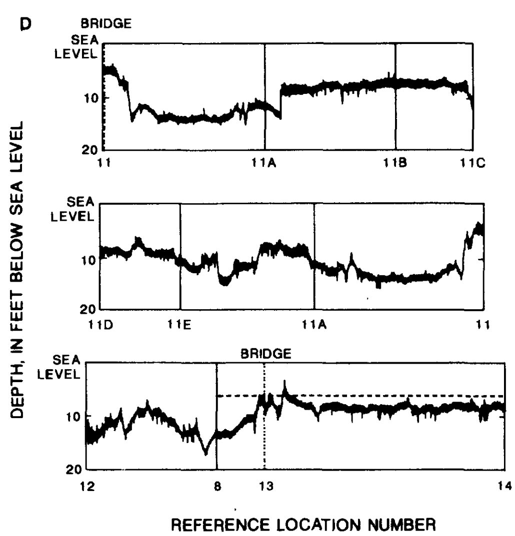
Figure 5. Path of fathometer survey in the Bluejay canal system.

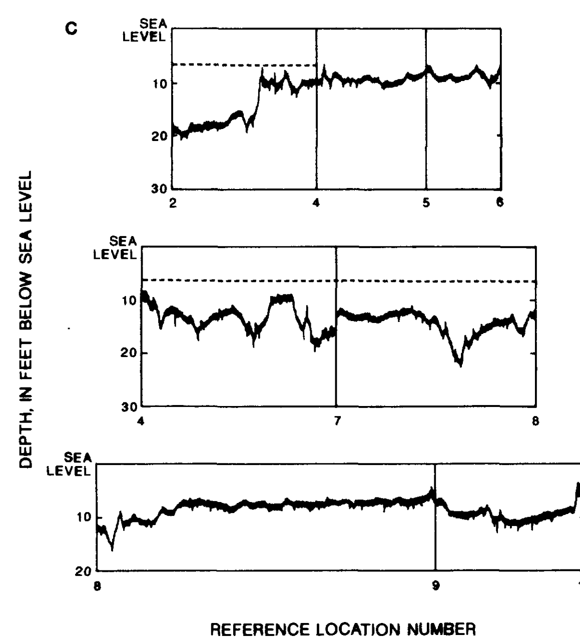
The canal reach from site 4 to the bridge at site 13 (fig. 5) has depths of up to 20 ft and is bounded by depths of 9 ft at site 4 and 6 ft at site 13. In this reach, measured depths average about 7 ft more than those given in the literature. Landward of site 13, the variation of canal depths is very gradual and generally within 1 or 2 ft of depths cited in the literature, except at dead ends such as at site 20 (fig. 5), where 2 or 3 ft of material has apparently accumulated on the canal bottom.

For purposes of this study, measured canal depths are used directly when available and indirectly when not available to guide depth assignments in other parts of the Bluejay canal system. Canal depths reported by Connell, Metcalf, and Eddy, Inc. (1979), are used throughout the San Carlos system.









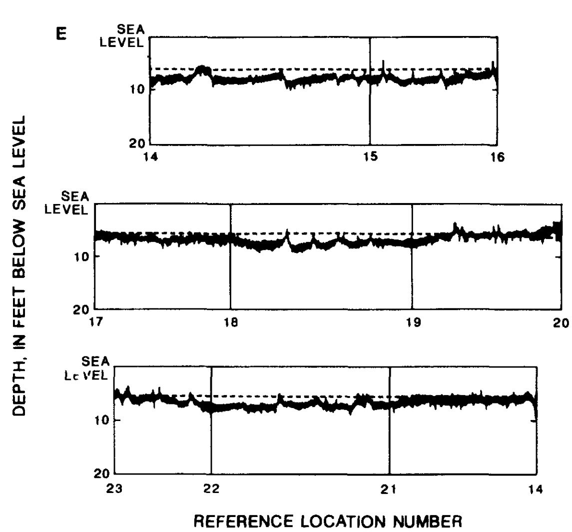


Figure 6. Graphs showing longitudinal depth profiles of selected canals in the Bluejay canal system.

In areas where possible tide-gate locations coincide with heavy accumulations of bottom sediment, preaccumulation depths also are used to allow for proper evaluation of tide-gate operation.

The term "lakes," as used in this report, applies to those parts of the study area that are significantly wider than the adjoining canals (fig. 4). The surface areas of lakes were scaled from recent maps of Cape Coral and range from 84.1 to 11.4 acres. The fathometer survey referenced in the previous section (fig. 5) included depth determinations in Lakes Finisterre, Thunderbird, and Britannia in the Bluejay system. Recorded depths along several lake transects are given in figure 6.

Depths in Lakes Thunderbird and Britannia generally range between 7 and 14 ft, but one deep hole of more than 20 ft was measured in the southwest quadrant of Lake Britannia (between locations 3C, 3J, and 3K, fig. 6B). Both lakes have bridges at their only outlet with depths of 6 and 5 ft, respectively. Depths in Lake Finisterre (sites 14, 21, 17, and 18, fig. 6E) are nearly constant at about 6 ft. For this study, measured depths were used for lakes in the Bluejay system, and depths reported by Connell, Metcalf, and Eddy, Inc. (1979), were used for the relatively small lakes in the San Carlos system.

Bridges and Other Constrictions

Constrictions in canal cross-sectional areas, such as at bridges, may influence water motion in the study area. There are four bridges and one other constriction in the Bluejay system and three bridges in the San Carlos system (fig. 4). One bridge in the Bluejay system is not shown because there is no water flow under this bridge during normal tidal conditions.

Field measurements were made to define the approaches and cross sections of three representative bridge sites and one cross section at a planned, but unconstructed, bridge site in the Bluejay system between fathometer sites 2 and 4 (fig. 5). Results of these surveys were used in the hydrodynamic simulation. The physical dimensions of bridges that were not surveyed were taken directly from information given in Connell, Metcalf, and Eddy, Inc. (1979).

Hydrologic Conditions

Hydraulic variables that were measured and evaluated for this study included tidal stage, tidal discharge, freshwater inflow, and density stratification. Additional variables affecting constituent transport, such as the concentrations of selected water-quality constituents and constituent dispersion characteristics, also were examined.

Tidal Stage

The periodic rise and fall of the tide is the primary cause of water motion in the study area and is the source of potential energy that may be tapped to improve water circulation and constituent flushing. Accurate tidal stage measurements at the existing tidal openings of the Bluejay and the San Carlos canal systems were required as boundary value input for hydrodynamic simulation. The accuracy of such simulations cannot exceed the accuracy of the input boundary conditions. Tidal stage measurements at other canal system sites provided a method for comparison of model-computed tidal stages with field-measured data as a means of calibration and to assess model performance and adequacy. Tidal-stage measurements also were made at locations selected for potential tidal openings to the study area.

A network of seven tide gages was established to measure tidal fluctuations at two existing and one possible new tidal opening to the study area, two sites within the Bluejay system, and two sites within the San Carlos system. Tidal-stage site locations are given in figure 7. Stage data-collection equipment included stilling wells and float-operated, digital instruments that recorded water-surface elevation every 15 minutes. The datum of each gage was referenced to sea level by spirit leveling to established reference monuments. Each gage was maintained during the study period and inspected at monthly intervals to assure reliable operation.

A comparison of three representative measured tides, one each at the mouths of the Bluejay and the San Carlos systems (fig. 7, sites Z1 and Z4) and another within the Rubicon system at a possible interconnection (fig. 7, site Z7), is shown in figure 8 for a typical tidal cycle. There is an obvious reduction in tidal range and a phase lag from the Bluejay system to both the San Carlos and the Rubicon systems. These differences are most apparent at low tide and are considered exceptionally large considering that the mouth of each canal system joins the same water body, the Caloosahatchee River, along a reach of only a few miles. The likely cause of the large tidal range and phase lag differences is the tidal-energy loss due to constrictions caused by extensive tidal flats and shoal areas near the mouth of the Caloosahatchee River. This energy loss is greater during low tide because of a sharply reduced river cross section that occurs with exposure of shoals. At high tide, the same phenomena occurs, but to a lesser degree, because submerged shoals provide more cross-sectional area, which results in less dissipation of energy.

The net result, however, is that tidal-stage differences of more than 0.5 ft frequently exist between the Bluejay and San Carlos systems and smaller stage differences exist between the San Carlos and the Rubicon systems. This indicates a potential to induce tidal flow between the canal systems in the study area. In both the Bluejay and the San Carlos canal systems, there is only a few minutes tidal phase lag between the mouth and head of each system indicating standing wave conditions and that the bridges and other constrictions have no significant effect on the tidal propagation.

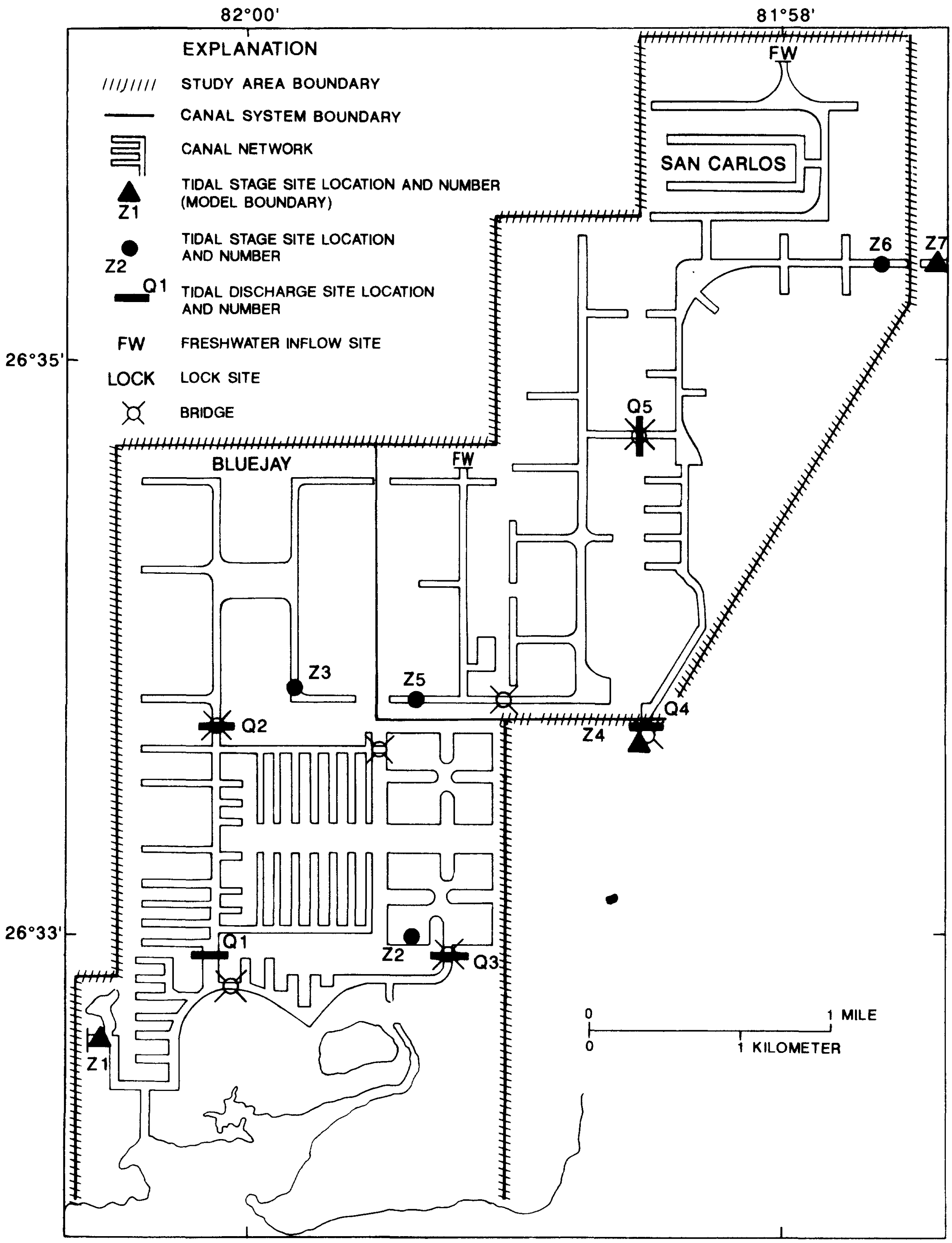


Figure 7. Location of tidal stage, tidal discharge, and freshwater inflow sites in the Bluejay and San Carlos canal systems.

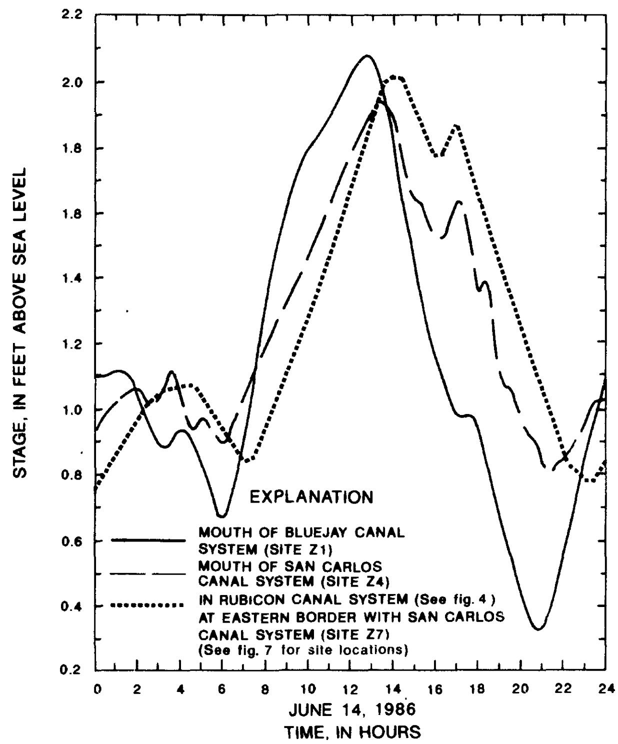


Figure 8. Measured tidal stage at the boundaries of the study area, June 19, 1986.

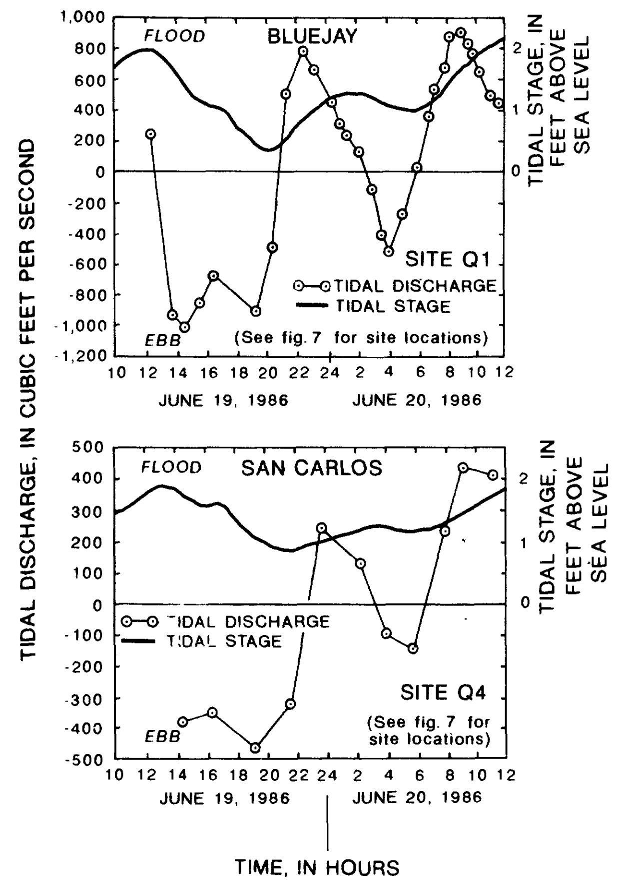


Figure 9. Measured tidal discharges in the Bluejay and San Carlos canal systems, June 19-20, 1986.

Tidal Discharge

A series of repetitive discharge measurements were made at two bridge sites and at one nonbridge site in the Bluejay system and at two bridge sites in the San Carlos system (fig. 7). The measurements covered a 24-hour period that started at about noon on June 19, 1986. The purpose of the discharge measurements was to assist in the calibration of and confirm the adequacy of the hydrodynamic simulation.

Each discharge measurement included velocity and cross-sectional area determinations in each of 15 to 20 subsections of the total canal cross section. Most measurements were made in 20 minutes or less and were judged to be representative of the average discharge during that time period. Two shifts of three two-person crews were used during a 24-hour period.

Discharge measurement results for the tidal outlets of the Bluejay and the San Carlos systems are given in figure 9 along with measured tidal stage. Measured discharges ranged from -1,020 to +910 ft^3/s at the mouth of the Bluejay system and from -460 to +440 ft^3/s at the mouth of the San Carlos system. Comparison between measured discharges and tidal stages confirms standing wave condition because the times of zero discharge, slack water, occur at times of tidal-stage peaks and troughs. Discharges measured in other parts of the systems show the expected progressive decrease in discharge magnitude with distance from the mouth.

An intense thunderstorm occurred in the study area from about 1600 to 1800 hours on June 19, 1986, and delivered an estimated 2 in. of rainfall. The runoff from this storm created an anomaly in the discharge data collected in the Bluejay system and in the tidal-stage data from both systems during this period. The tidal stage stopped falling for about an hour during the storm with a corresponding measured decrease in ebb discharge in the Bluejay system. A similar condition probably also occurred in the San Carlos system but is not as readily apparent from the available measurements. Apparently, the combination of strong winds and stormwater runoff was sufficient to offset the falling tide for a short period, which resulted in ebb discharge reductions.

Freshwater Inflow

The sources of freshwater inflow to the Bluejay and the San Carlos canal systems are direct rainfall input to open water surfaces, overland runoff from adjacent land areas, and seepage from the surficial aquifer into which the canals and lakes have been dredged. In addition, the San Carlos system receives some inflow from the extensive upstream freshwater canal system through two broad-crested weirs (fig. 7). There is no inflow to the Bluejay system from the freshwater canals.

Discharge from the freshwater canal system is controlled by a series of 16 weirs with crest elevations ranging from 2.0 to 6.0 ft (Fitzpatrick, 1986, p. 7). Most of the discharge is routed westward toward Matlacha Pass through eight control structures with crest elevations ranging from 2.0 to 2.4 ft. In contrast, crest elevations of the two weirs on the boundary of the San Carlos system are 3.0 and 4.0 ft. Freshwater inflow from this source was estimated to range from zero to about 50 ft^3/s during times of visual observation and less than 10 ft^3/s during intensive data collection in June 1986. This inflow provides some flushing of the San Carlos system, particularly during high runoff conditions common during late summer and early fall. For most of the year, however, inflows from the freshwater canal systems are relatively low (Goodman, 1983) and can be considered negligible in comparison with the magnitude of the tidal exchange.

The rate of freshwater seepage from the surficial aquifer to the Bluejay and the San Carlos canal systems is not known. Some potential for seepage does exist, however, because water levels in the surficial aquifer within the study area were reported by Fitzpatrick (1986, p. 7) to be from 0.55 to 4.26 ft above sea level during the dry season in May 1980. For purposes of this study, however, the rate of seepage is presumed to have a negligible effect on tidal flow and circulation.

Rainfall on canal and lake surfaces and stormwater runoff from the adjacent land during intense storm events is known to have short-term effects that can significantly modify normal tidal water motion in the study area. The effects of one such event that occurred on June 19, 1986, was briefly described in the previous section. The long-term effect of these events on canal flushing is not well understood, but it is apparently insufficient to prevent the development of water-quality problems in dead-end canals. It is expected that runoff events will have much the same effect on the canal systems regardless of modifications by the addition of tide gates.

Analysis of the sources of freshwater into the study area generally indicates that the effects on tidal flow, circulation, and flushing are either negligible, of short duration, or expected to be similar with or without possible modifications. The assumption of no freshwater inflow was therefore considered valid for the purposes of this study, which emphasizes the dominant, tide-induced effects on the canal systems.

Water Quality

Samples were collected during the study from three separate water-quality networks that were established to meet three different objectives (fig. 10). The first network consisted of the seven tidal-stage sites and was established to define general water-quality conditions and the seasonal variability of temperature, specific conductance, and dissolved oxygen. The objective of the second network, sampled June 19-20, 1986, was to define the vertical and longitudinal variability of nutrient concentration and specific conductance at high, mid, and low tides in representative primary, secondary, and tertiary canals. A third network was sampled February 24-25, 1987, to investigate water quality in deep areas, at canal extremities, in lakes, and in the Caloosahatchee River to help evaluate the probable effects of tide gates on canal water quality.

General water-quality conditions in the study area were determined by measurement of water temperature, specific conductance, and dissolved oxygen at the seven tidal-stage sites from November 1985 to June 1987. Observations were made at two or more depths in the water column at approximately monthly intervals. Water temperature, specific conductance, and dissolved oxygen variation for six of these sites, three in the Bluejay system and three in the San Carlos system are shown in figures 11 and 12, respectively. In both figures, measurements at the top and bottom of the water column are shown where significant differences existed between the two, otherwise only the top measurements are given.

Water temperatures measured during the study ranged from about 17 to 32 °C. The water temperatures exhibited a marked seasonality with low temperatures from December to March and high temperatures usually in September. In general, surface temperatures tended to be slightly warmer than bottom temperatures, particularly during the summer.

Specific conductance was more spatially variable than temperature. Maximum conductances at sites closest to the source of saltwater (fig. 10) ranged from 40 to 45 mS/cm during low rainfall and runoff periods, whereas sites closer to sources of freshwater (fig. 10, sites Z5 and Z6) had maximum conductances ranging from 30 to 33 mS/cm during the same time period. Low specific-conductance values (0.7-15 mS/cm) were measured throughout the study area during September 1986 as a result of high local rainfall and relatively high Caloosahatchee River discharge from June to September, as shown in figure 3. At one site, Z5, low specific conductance persisted into November 1986 due to continued local freshwater inflow to the San Carlos canal system near this site. The greatest vertical difference in specific conductance was found at site Z1 during periods of high specific conductance from January to March 1986. The difference ranged from 5 to 9 mS/cm and probably resulted from releases of freshwater at the lock structure just upstream from the sampling site.

All dissolved-oxygen measurements (figs. 11 and 12) were collected during daytime hours and are therefore probably influenced by oxygen production due to photosynthesis. In September 1986, dissolved-oxygen

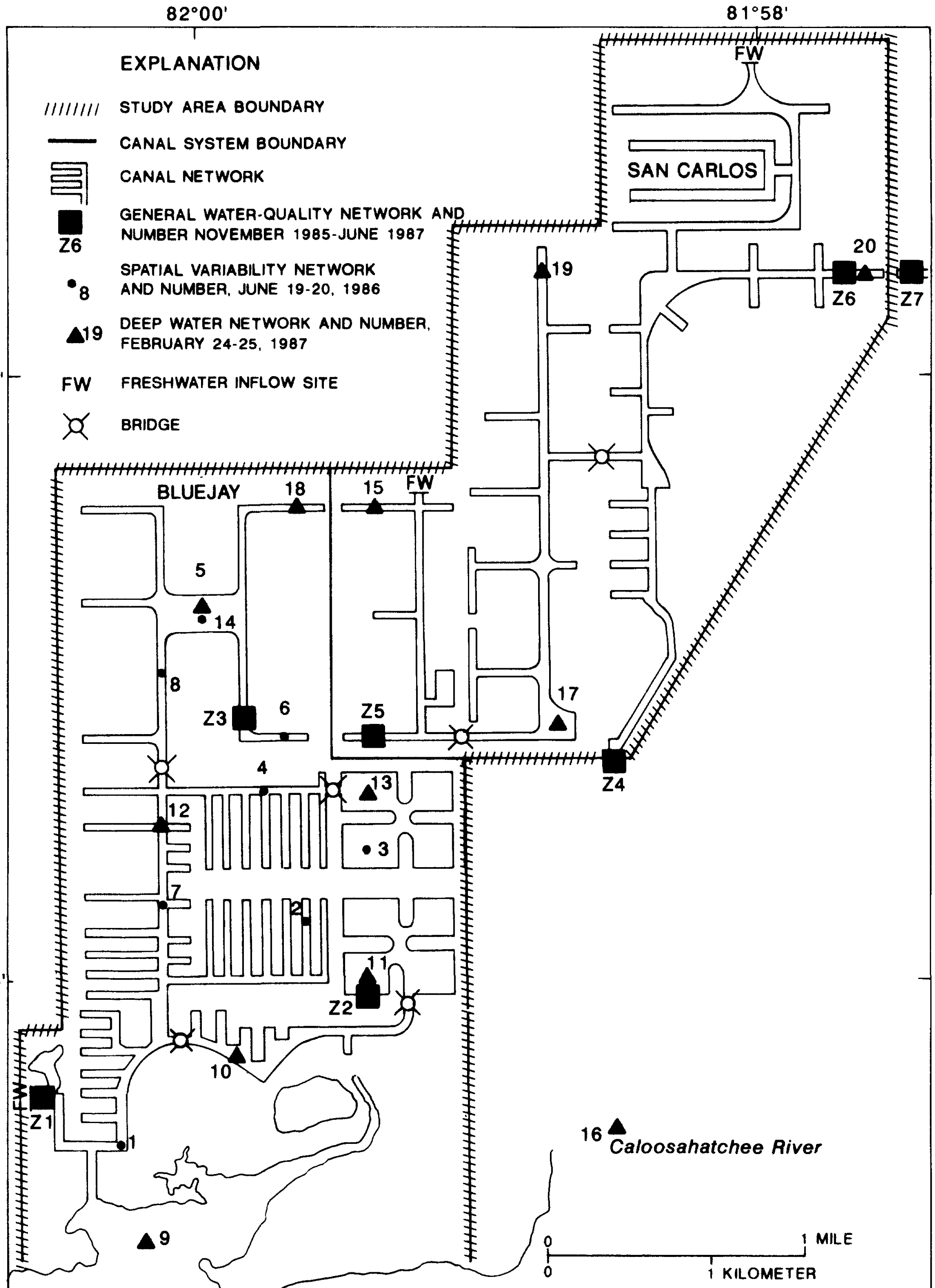


Figure 10. Water-quality sampling sites.

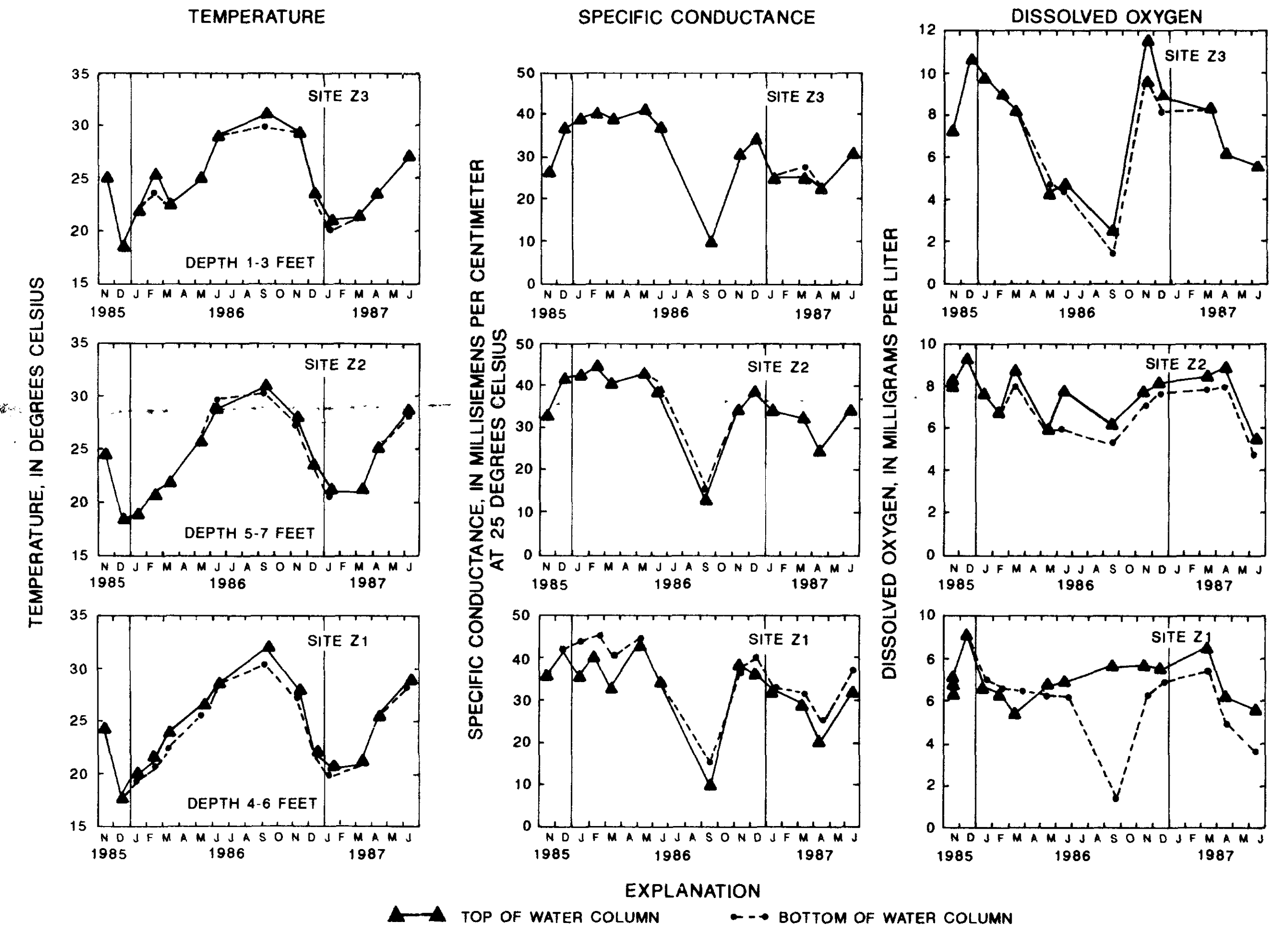


Figure 11. Temperature, specific conductance, and dissolved-oxygen variation in the Bluejay canal system, November 1985-June 1987.

concentrations less than the State minimum standard of 4.0 mg/L were measured at two sites in the Bluejay system, a bottom reading at site Z1 and top and bottom readings at site Z3. A bottom reading at site Z1 in June 1987 also was less than 4.0 mg/L. Site Z3 displayed the greatest range in measured dissolved-oxygen concentrations, from a low of about 2 mg/L associated with the high rainfall period of September 1986 to a high of about 12 mg/L in November 1986. In general, measured dissolved-oxygen concentrations in both the Bluejay and San Carlos systems were lowest during times of high water temperature and low specific conductance. This reflects the reduced solubility of oxygen in water with increased temperature and probable greater oxygen demand from materials carried to the canals from freshwater runoff. At most sites, water near the bottom tended to contain less dissolved oxygen than water near the top during the study period. This is an indication that the rate of reaeration at the water-air interface, oxygenation by photosynthesis, and mixing within the water column are insufficient to supply oxygen to the bottom water at the rate oxygen is being depleted at the bottom. This vertical difference in dissolved oxygen was not observed at site Z4, an area of high tidal velocity and accompanying turbulence, nor for much of the time at site Z3 where the water is shallow and bottom processes appear to dominate the entire water column.

Additional water-quality samples were collected in the Bluejay canal system June 19-20, 1986, to determine the degree of stratification and the vertical and longitudinal distribution of several nutrient species. Unfiltered top and bottom samples were collected at sites 1 through 8, shown as circles in figure 10, and analyzed at U.S. Geological Survey laboratories for specific conductance, ammonia, organic nitrogen, nitrite, nitrate, orthophosphate, and total phosphorus. In an attempt to avoid a bias that could be introduced by sampling at only one time during the tidal cycle, at most of the sites, samples were collected at high, low, and midtidal stages. At sites 7 and 8, however, samples were collected only at high tide.

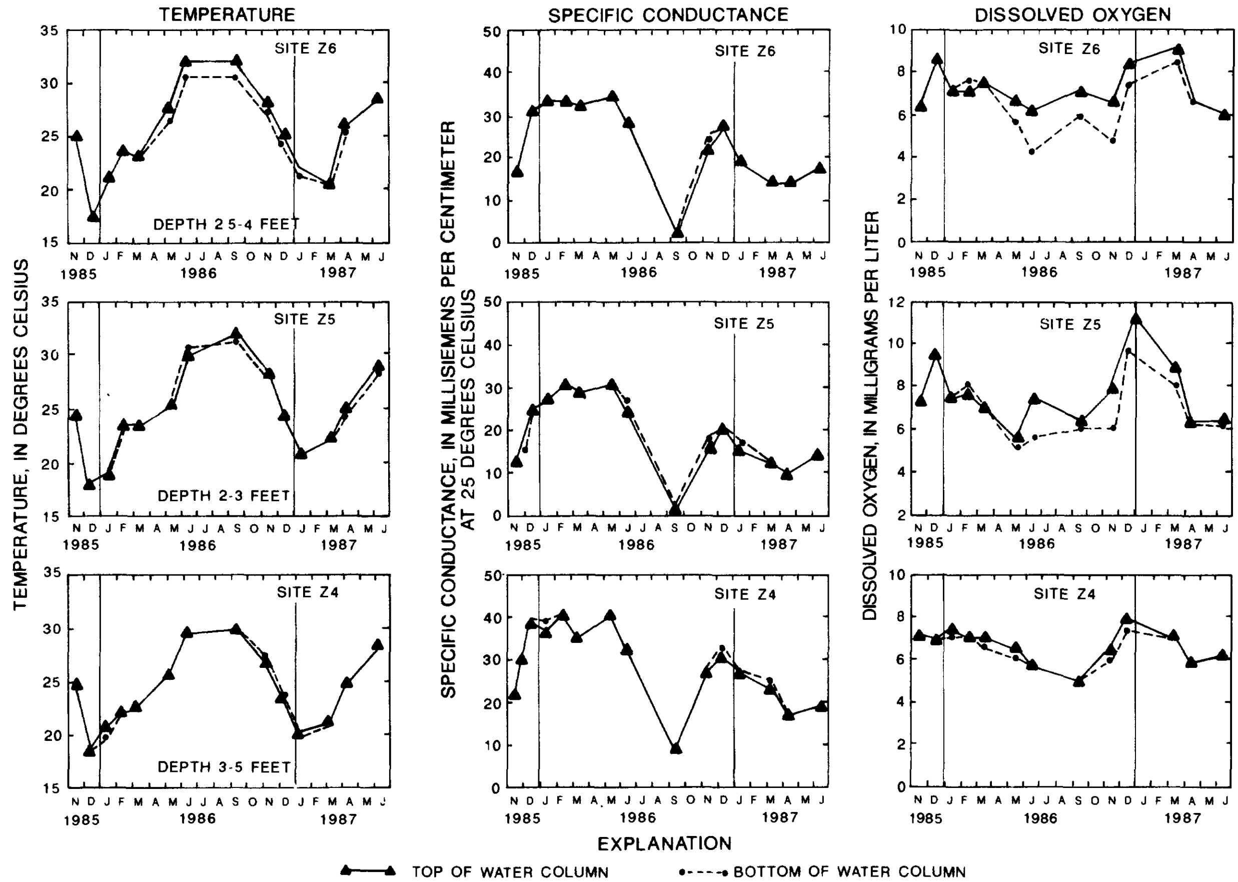


Figure 12. Temperature, specific conductance, and dissolved-oxygen variation in the San Carlos canal system, November 1985-June 1987.

The analytical results for these samples are presented in table 1 in summary form. For sites 1 through 6, the maximum, minimum, and mean for each measured variable are given for the top and bottom of the water column. Single-sample results are given for sites 7 and 8. No results are shown for nitrite or nitrate as all values were at or very near the analytical detection limit.

The data indicate that the canal system is partially to well stratified with the difference in specific conductance between top and bottom waters ranging from less than 3 mS/cm at sites 4 and 6 to greater than 11 mS/cm at sites 2 and 7. In general, nutrients were slightly more concentrated in bottom waters throughout the system, with the exception of site 1 where higher organic nitrogen and orthophosphate concentrations occurred at the surface. No obvious large spatial differences in nutrient content between different parts of the system, however, were observed.

A more areally extensive survey of water quality in the study area was made February 24-25, 1987, and included samples from both the Bluejay and San Carlos systems. In addition, samples of Caloosahatchee River water also were collected near the entrances of the canal systems. This survey was undertaken to characterize water-quality conditions in the deeper parts of the canals, canal extremities, lakes, and the Caloosahatchee River and to provide a basis for estimating water-quality changes that may be induced by construction of tide gates. The locations of the sites sampled, numbered 9 through 20, are shown as triangles in figure 10. Samples were collected at middepth (for depths of less than 5 ft), at top and bottom (for depths between 5 and 10 ft), and top, mid, and bottom (for depths greater than 10 ft) and analyzed for ammonia, organic nitrogen, nitrite, nitrate, orthophosphate, and total phosphorus. Field measurements of water temperature, specific conductance, pH, and dissolved oxygen also were made.

The primary results of this survey are summarized in figures 13 and 14. Nitrite and nitrate are omitted because concentrations for all samples were at or very near the analytical detection limit. In figure 13, the site number, date and time of sample collection, and all significant sampling results are shown clustered near the depth point to which they apply. Viewed in this way, the influence of depth on constituent distribution is more apparent. Specific conductance ranged from 9.4 to 38.0 mS/cm and generally increased with depth.

Table 1. Summary of specific conductance and nutrient data collected in the Bluejay canal system, June 19-20, 1986

[mg/L, milligrams per liter; mS/cm, millisiemens per centimeter]

Constituent	Site 1 Depth, 15 feet		Site 2 Depth, 10 feet		Site 3 Depth, 10 feet		Site 4 Depth, 8 feet	
	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
NH4-N (mg/L)								
Mean	0.03	0.08	0.04	0.08	0.04	0.05	0.04	0.04
Range	0.02–0.04	0.03–0.12	0.03–0.04	0.03–0.12	0.03–0.04	0.03–0.05	0.03–0.04	0.04–0.05
Organic N (mg/L)								
Mean	0.87	0.75	0.77	0.82	0.80	0.84	0.68	0.86
Range	0.76–0.93	0.72–0.77	0.73–0.79	0.72–0.90	0.74–0.84	0.82–0.88	0.60–0.73	0.79–0.94
PO4-P (mg/L)								
Mean	0.09	0.06	0.05	0.10	0.07	0.07	0.05	0.07
Range	0.08–0.10	0.04–0.08	0.04–0.07	0.06–0.13	0.07–0.07	0.05–0.10	0.05–0.05	0.05–0.10
Total P (mg/L)								
Mean	0.14	0.14	0.09	0.16	0.10	0.17	0.09	0.12
Range	0.12–0.15	0.13–0.16	0.08–0.12	0.15–0.17	0.10–0.11	0.15–0.19	0.08–0.10	0.10–0.16
Specific conductance (mS/cm)								
Mean	29.4	36.5	29.9	41.8	39.3	43.0	35.0	37.6
Range	28.1-31.2	32.5-40.0	27.2-33.5	41.6–42.1	39.3–39.5	42.7–43.2	34.0–36.0	36.0–40.4

Constituent	Site 5 Depth, 6 feet		Site 6 Depth, 5 feet		Site 7 Depth, 15 feet		Site 8 Depth, 7 feet	
	Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
NH_4-N (mg/L)								
Mean	0.02	0.04	0.03	0.07	0.04	0.12	0.03	0.05
Range	0.02–0.03	0.04–0.06	0.03–0.03	0.03–0.11	—	—	—	—
Organic N (mg/L)								
Mean	0.60	0.81	0.78	0.92	0.66	0.90	0.55	0.72
Range	0.55–0.64	0.71–0.99	0.75–0.80	0.79–1.15	—	—	—	—
PO_4-P (mg/L)								
Mean	0.03	0.05	0.04	0.06	0.07	0.08	0.04	0.05
Range	0.02–0.04	0.05–0.06	0.02–0.05	0.05–0.07	—	—	—	—
Total P (mg/L)								
Mean	0.06	0.12	0.08	0.13	0.12	0.20	0.10	0.12
Range	0.05–0.07	0.11–0.13	0.07–0.09	0.10–0.15	—	—	—	—
Specific c onductance (mS/cm)								
Mean	27.8	36.6	31.7	34.4	26.6	38.1	29.4	35.5
Range	25.5–29.3	35.7–37.5	30.6–32.4	32.2–36.0	—	—	—	—

Dissolved-oxygen concentrations ranged from 2.0 to 7.9 mg/L and generally decreased with depth. Temperature ranged from 20.0 to 23.5 °C, and pH ranged from 6.8 to 7.9. Similarly, the distribution of ammonia, organic nitrogen, orthophosphate, and total phosphorus are presented in figure 13.

Histograms of analytical results for five constituents are shown in figure 14. All of the histograms show similar skewed distributions with most observations near one end of the horizontal axis and progressively fewer observations toward the other end. For purposes of this study, the median value is used as an appropriate measure of central tendency with an acceptable central range of twice the difference between the median and lowest concentration for nutrients and twice the difference between the median and highest concentration for dissolved oxygen. Nutrient and

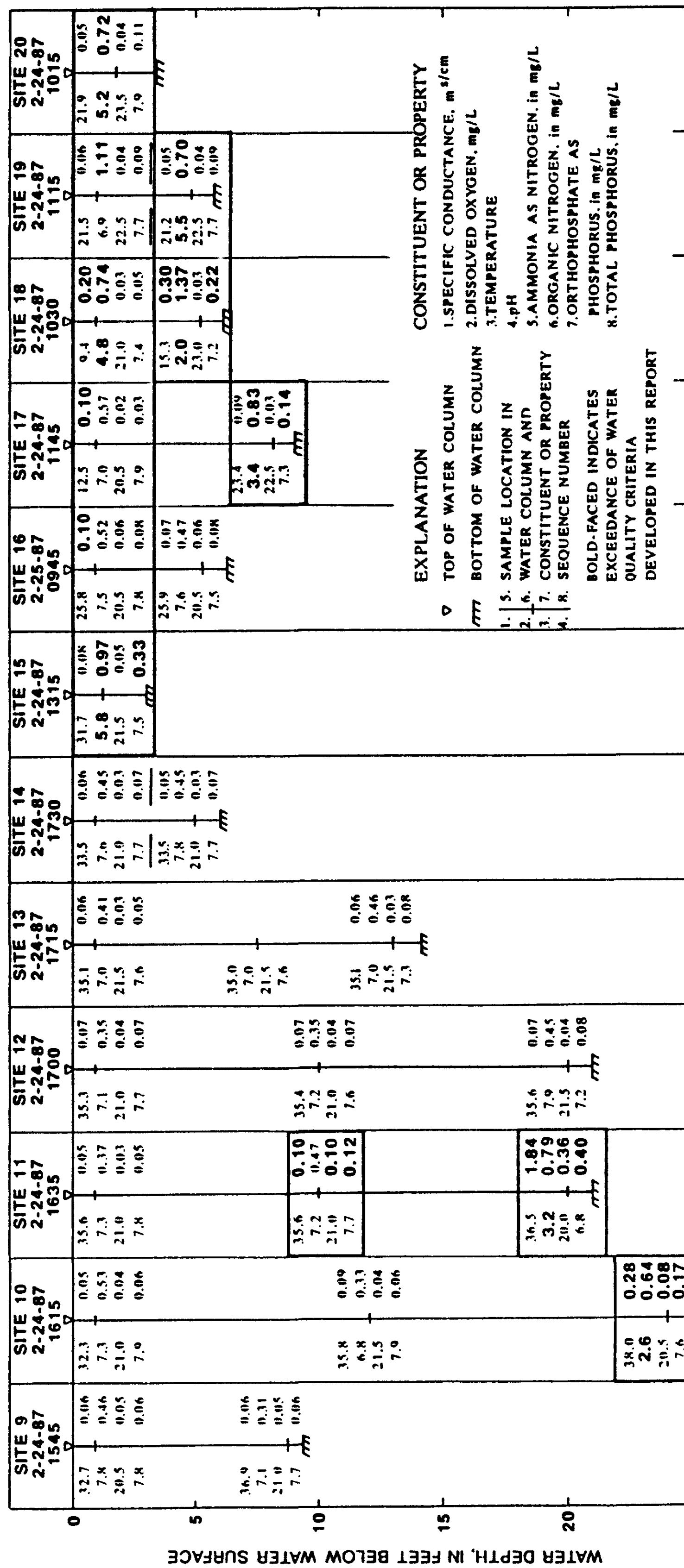


Figure 13. Schematic showing summary of water-quality variation with depth, February 24-25, 1987.

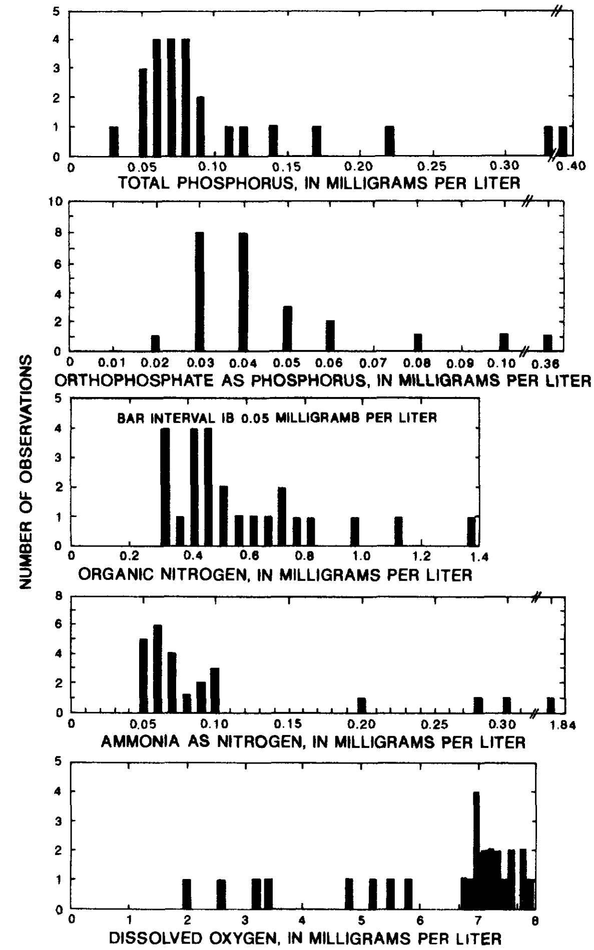


Figure 14. Histograms of selected water-quality constituents measured on February 24-25, 1987, at selected sites in the Bluejay and San Carlos canal systems and in the Caloosahatchee River.

dissolved-oxygen concentrations outside of this central range are considered to be outliers and are indicative of undesirable water-quality conditions relative to the bulk of the observations. Observations identified as indicating potential water-quality problems are shown in bold-face type in figure 13. Analyses containing one or more outlying observations are enclosed in squares that identify their location and depth. The table shown below gives the computed mean, median, and central range for each of the five constituents; summarizes the numerical criteria for outlying data established for purposes of this report; and gives the number of observations identified as outliers for each of the five variables.

The results of this analysis led to several conclusions. Inspection of figure 13 reveals that more than one bold-faced number often appears in the same sample or same site. Although low dissolved oxygen is not expected during the cool winter months, two sites in each canal system had measured values less than the State standard of 4.0 mg/L. Sites 10 and 11 in the Bluejay system (fig. 10) are in areally extensive deep water that has intervening shallow sills to the Caloosahatchee River (see figs. 5 and 6). This deep water is apparently sufficiently stagnant to produce low dissolved-oxygen or perhaps even anoxic conditions. The elevated nutrient levels are probably associated with anoxic processes that occur at or near the bottom of these deep basins. The most revealing aspect of the outlier distribution (other than at sites 10 and 11) is that undesirable conditions are limited to sites with relatively low specific conductance (less than 32 mS/cm) and that the most degraded site (site 18) has the lowest specific conductance. This indicates that undesirable water-quality conditions are found where freshwater inflow is not sufficiently dispersed with higher conductivity river water.

Dispersive Characteristics

Dissolved and suspended constituents in the study area are transported within the canals by both advective and dispersive processes. Advection implies the common notion of material being moved with the mean velocity of the water. Dispersion can be considered as mixing process that accounts for the differing advective velocities of the water in the cross section.

In dead-end tidal canals, advection by tidal action alone cannot produce constituent flushing because the nature of advection is oscillatory, moving material alternately out and then back in with no net outward motion. Dispersion associated with tidal motion, however, causes a mixing or spreading of transported material, which results in a net outward movement, or flushing.

Variable	Number of observa-tions	Mean[1]	Median[1]	Central range[1]	Criteria[1]	Number of outlying observa-tions
Dissolved oxygen	25	6.5	7.0	6.1 –7.9	<6.1	8
Ammonia-N	25	.16	.07	0.05–0.09	>.09	7
Organic nitrogen	25	.59	.47	0.31–0.63	>.63	9
Orthophosphate-P	25	.06	.04	0.02–0.06	>.06	3
Total phosphorus	25	.11	.08	0.03–0.13	>.13	5

[1]Units are in milligrams per liter.

Introduction of tide gates into the study area is expected to increase advective flushing in those canals that are affected directly. Flushing in other canals would be limited to the dispersive mechanism with one important difference. Material dispersing from the canals would have to be transported a shorter distance before encountering an advecting canal. Once this happens, the dispersed material could be flushed from the system more rapidly.

Simulation of advective constituent transport does not require additional information to that provided by the results of hydrodynamic modeling. Simulation of dispersive constituent transport, however, requires assignment of a dispersion coefficient that is best determined by a combination of field tracer measurements and application of a numerical routing procedure described by Jobson and Schoellhamer (1987).

Determination of representative dispersion coefficients using slug injections of Rhodamine WT dye at three sites within the Bluejay system was attempted June 16-20, 1986. Equipment used for this field operation included four specially designed floating monitors with 24 spring-loaded syringes for automatic collection of water samples, as described by F.A. Kilpatrick (U.S. Geological Survey, written commun., 1971). A boat-mounted, flow-through fluorometer with an attached data logger also was used. The boat-mounted equipment was used to traverse the canals and provide information on dye concentrations between the anchored floating monitors.

Unfortunately, the effects of a large thunderstorm that moved into the study area shortly after dye injection rendered the data from this effort unusable. The primary interference was rapid stormwater runoff that probably led to formation of a freshwater layer on top of the saltwater into which the dye had been injected. The floating equipment was only capable of collecting samples from the freshwater layer that contained little or no dye. Another complication noted during the stormwater event was the reversal of surface currents from incoming to outgoing with a continuous rise in tidal stage. These conditions, along with the possibility of other interferences, such as turbidity supplied by stormwater and an apparent bloom of fluorescing plankton, negated the intended use of the dye-dispersion data.

Despite the difficulties encountered in the interpretation of the measured dye data, model application was achieved using dispersion coefficients reported in the literature for similar systems. The effects of variations in longitudinal dispersion coefficients on simulation results was also investigated. The available data indicated that dispersion coefficients should be on the order of 1.0 ft^2/s and vary by less than one order of magnitude (U.S. Environmental Protection Agency, 1975).

HYDRODYNAMIC MODEL DEVELOPMENT FOR THE EXISTING CANAL SYSTEM

To evaluate the potential effects of the installation of tide gates on the circulation and flushing characteristics of the study area, numerical simulation of tidal hydrodynamics was undertaken. Flow simulations were accomplished using a one-dimensional, branched-network flow model, BRANCH, that is described in detail by Schaffranek and others (1981). This model has been extensively and successfully applied by both the U.S. Geological Survey and other investigators to a wide range of unsteady flow situations in the United States and other countries (Holtschlag, 1981; Stedfast, 1981; Jennings and Jeffcoat, 1987; and Bergquist and Ligteringen, 1988). BRANCH was chosen for this application because it is a well-documented, proven model that has the requisite features or sufficient adaptability to accommodate the needs of this study.

Governing Equations and Assumptions

The partial differential equations of motion and continuity that describe unsteady, one-dimensional flow in open channels have been presented by Baltzer and Lai (1968) and are reproduced here in the form given by Schaffranek and others (1981). The equation of continuity is

$$B\frac{\partial Z}{\partial t}+\frac{\partial Q}{\partial x}=0 \qquad (1)$$

in which the dependent variables are

Z, the water-surface elevation, and
Q, the channel discharge.

The independent variables are

B, the channel top width,
x, the longitudinal distance, and
t, the elapsed time.

The equation of motion used in the BRANCH model is

$$\frac{1}{gA}\frac{\partial Q}{\partial t}+\frac{2\beta Q}{gA^2}\frac{\partial Q}{\partial x}-\frac{\beta Q^2}{gA^3}\frac{\partial A}{\partial x}+\frac{\partial Z}{\partial x}+\frac{k}{A^2R^{4/3}}Q\,|Q| - \frac{\xi B}{gA}U_a^2\cos\alpha=0 \qquad (2)$$

where

g is the acceleration of gravity,
A is the cross-sectional area,
R is the hydraulic radius, and
k is a flow-resistance coefficient related to η (similar to Manning's n in steady-flow conditions) by $k=\left(\frac{\eta}{1.49}\right)^2$ in the inch-pound system of units or $k=\eta^2$ in the metric system,
U_a is the wind velocity making an angle, α, with the positive x-axis, and

ξ is a dimensionless wind-resistance coefficient defined as

$$\xi = C_d \frac{\rho_a}{\rho},$$

where

C_d, is the water-surface drag coefficient,

ρ, is the water density, and

ρ_a is the air density.

The momentum, or Boussinesq coefficient, β, is included in the equation of motion to account for the nonuniformity of water velocity throughout the cross section of open channels. In highly nonuniform flows, the momentum effects of these variations can be taken into account by this coefficient defined by conservation of momentum principles as

$$\beta = \frac{\int u^2 dA}{U^2 A}$$

where

u represents the velocity of water flowing through some finite area, dA, and

U is the average velocity in the total channel cross section, A.

Hydrodynamic assumptions inherent in the derivation of these one-dimensional equations of motion and continuity must be considered to initially determine whether the model is appropriate for application. Model results must then be qualified based on the degree to which the assumptions are met. Assumptions from the derivation process include (from Schaffranek and others, 1981):

- The slope of the channel bottom is mild and reasonably constant over the reach being simulated and subcritical flow is maintained.
- Lateral flow into or out of the channel is negligible between channel junctions.
- The Manning formula is an adequate representation of frictional resistance in an unsteady flow environment.
- Water density in the flow system is essentially homogeneous with a hydrostatic pressure distribution throughout.
- A uniform velocity distribution exists in each cross section, except as can be accounted for by use of the momentum coefficient, β.

The Bluejay and the San Carlos canal systems are considered nearly ideal for application of the BRANCH model. All canal sections are either straight or gently curving with box-cut cross sections, and most have uniform or gradually varying depths, flow velocities well within the subcritical range and flat or gradually varying bottom slopes. The cross-sectional areas are well defined, with no overbank or flood-plain sections having complex flow-resistance characteristics. Sufficient data were collected during this study to adequately define tidal-stage conditions at the model boundaries and tidal stage and tidal discharge within each system for model calibration and verification.

Some attributes of the study area, however, are not ideal for application of any one-dimensional model. The various lakes certainly have flow patterns that are only crudely approximated by the one-dimensional assumption. As was observed during the study, there are times when tidal flow in the canals is interrupted by large runoff events that apparently produce density-stratified conditions that do not meet the one-dimensional assumption. Flow from the upstream fresh-water canal system also can, on occasion, produce density stratification in the canals of the San Carlos system. In spite of these recognized limitations of the model in some locations and at certain times, application of the BRANCH model to the study area is considered appropriate and the overall results are considered reliable. Overinterpretation of results in the lakes and during times of stratification, however, should be avoided.

Numerical Methods

Approximate solutions to the partial-differential equations of motion and continuity used by the BRANCH model (eqs. 1 and 2, respectively) are obtained by use of appropriate finite-difference techniques, as described in detail by Schaffranek and others (1981) and briefly summarized here. In general, this involves substituting a grid system having multiple segments of finite length and multiple time steps of a constant, finite period for the length and time derivatives, respectively, in the partial-differential equations. A generalized schematic representation of a canal system showing a BRANCH network type grid and giving the terminology used for various model sections and points is given in figure 15.

The BRANCH model uses a weighted, four-point, implicit, finite-difference scheme that treats time derivatives of stage and discharge as centered in both time and space. Spatial derivatives of stage and discharge are centered in space and positioned in time according to a user defined factor, and the spatial derivative of the cross-sectional area in the equation of motion is represented by a forward-difference approximation. Geometric properties of area, top width, and hydraulic radius (approximated by hydraulic depth), as well as the nonderivative discharge in the equation of motion, are treated as centered in space and positioned in time according to a second user-defined factor.

The value of the two user-defined weighting factors have effects, discussed by Schaffranek and others (1981), on computational stability. Variation of these factors between 0.6 and 1.0 in this study was found to produce a maximum difference of 2 percent in computed stage and discharge. A value of 0.7 for both factors was used in this study and found to produce stable results.

The BRANCH model uses a transformation procedure to efficiently solve the large set of algebraic equations that result from application of the four-point, implicit, finite-difference scheme. It is possible to solve these equations directly for stage and discharge at the ends of all individual

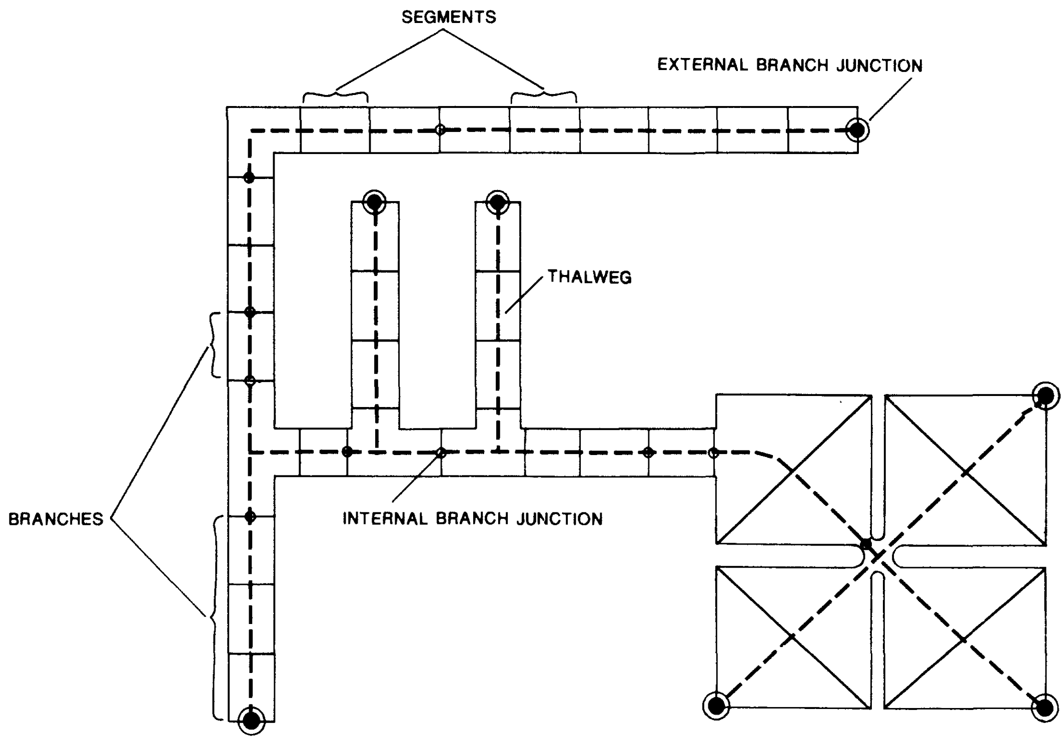


Figure 15. Generalized schematic of a modeled canal network.

segments, as defined in figure 15. The transformation procedure, however, reduces computational effort by nearly an order of magnitude. Initially, stage and discharge relations are defined at segment locations between branch junctions in terms of stage and discharge at the junctions themselves. A Gaussian elimination technique that uses maximum pivot strategy is then applied to provide a matrix solution for stage and discharge at branch junctions. With the previously established relations, these junction results are substituted to determine stage and discharge at each intervening segment between branch junctions.

Schematization of the Canal Systems

The study area was schematized according to the general form shown in figure 15 using 80 branches and 73 junctions and a total of 225 segments varying from 2 to 8 per branch. Information defining the length of each segment, as well as the variation of both cross-sectional area and top width over the range of tidal stage, was input to the model at the ends of each segment. Segments were carefully positioned to account for all significant cross-sectional area changes within the canals, including bridges and lakes.

Bridges generally were defined by three segments representing the two converging approaches to the bridge and the throat or bridge section itself. Lakes were defined in various ways to preserve their individual hydraulic characteristics. Lake Finisterre (fig. 5), for instance, was simply represented as one wide canal segment. Lake Britannia and Lake Thunderbird (fig. 5), however, were each represented as four diamond-shaped quadrants requiring two segments to define each quadrant.

To make efficient use of computer resources, many individual canals were not directly represented in the model. The effect of nonschematized canals was accounted for indirectly by lumping several parallel canals together into one wide branch within the model. In this way, the surface area, cross-sectional area, width, and volume of the canals were adequately represented but with a significant saving of computative effort. The effect of canal lumping was tested in separate model comparisons designed specifically for that purpose, and no differences in stage and velocity computations between complete canal representation and canal lumping were detected.

Potential tide-gate locations are included within the model at sites shown in figure 16. The width of each gate is assumed to be 10 feet and the water depth is assumed to be 5 ft at sea level. During model calibration and verification, all tide gates were kept in their closed position to represent existing conditions in the study area.

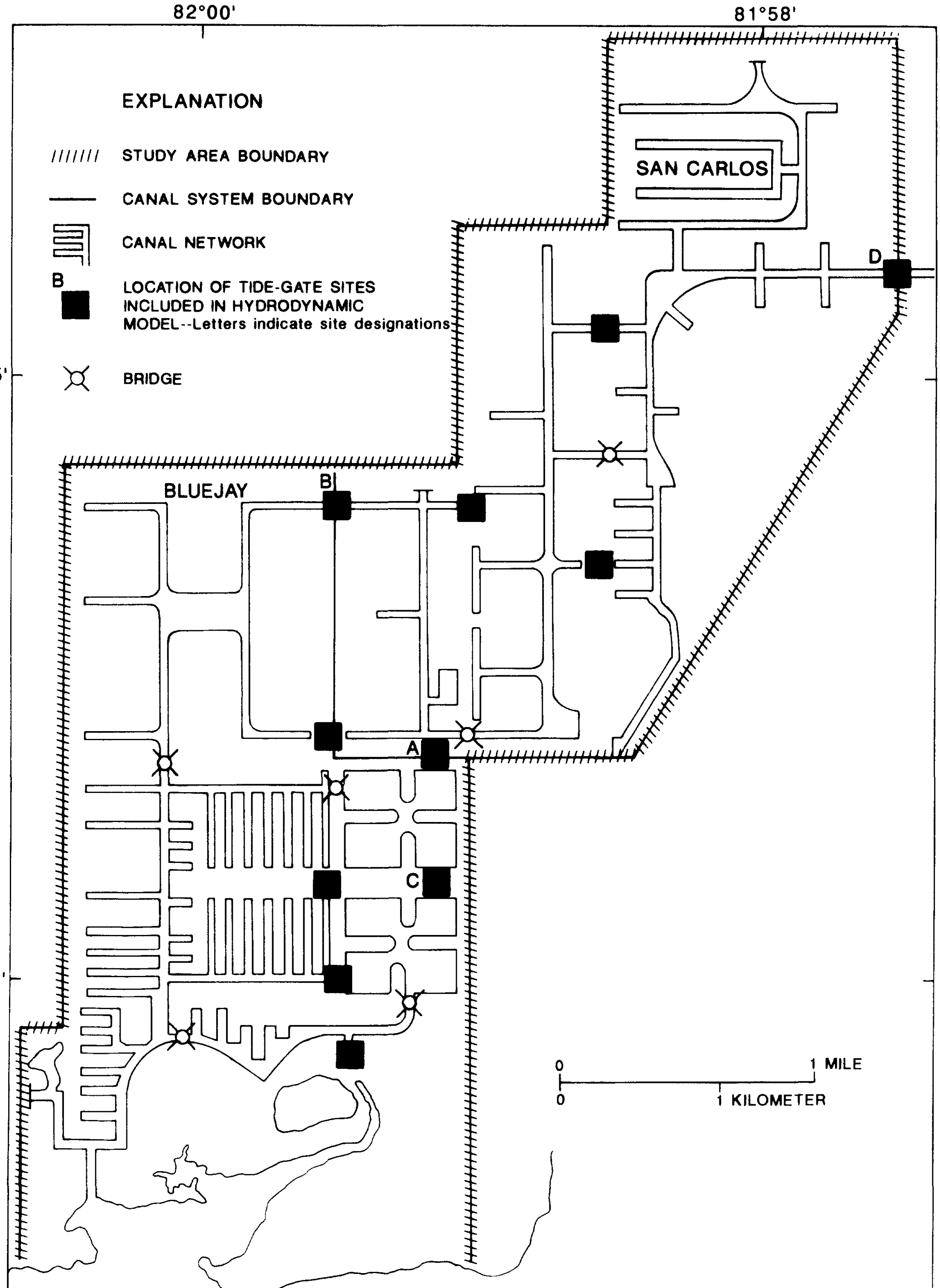


Figure 16. Locations of modeled tide-gate sites.

Model Calibration and Verification

Calibration and verification are necessary steps in model application. During calibration, the flow-resistance coefficients that are not measurable in the field are adjusted so that model computations match one set of field measurements as closely as possible. During verification, the degree of similarity between the real system and model computations is tested by comparing computed results with another set of field measurements without changing any model coefficients.

The calibration data set consisted of tidal-stage measurements made at four sites within the study area during June 19-20, 1986. Two of the sites (Z2 and Z3) are in the Bluejay system and two are in the San Carlos system (Z5 and Z6), as shown in figure 7. The verification data set consisted of tidal stage at the same sites during June 16-18, 1986, as well as discharge measurements at five sites, also shown in figure 7, during a 24-hour period starting about noon on June 19, 1986. The discharge data are considered the most sensitive of the field measurements, so they were retained specifically for verification purposes. For all computations, tidal-stage data were supplied for the boundary sites shown in figure 7 and freshwater inflow was assumed to be zero.

Model computations of tidal stage were found to be only mildly sensitive to changes in the flow-resistance coefficient. Successive calibration runs, using boundary stage data from June 19-20, 1986, were made with spatially constant flow-resistance coefficients of 0.023, 0.025, 0.026, and 0.029. A flow-resistance coefficient of 0.026 was found to produce the best comparison between computed and measured stage. The standard error of calibration at the four internal tide-gage sites ranged from 0.013 to 0.033 ft with an average of 0.026 ft. For comparison, the tidal range at the four internal tide gages averaged 1.50 ft on June 19 and 20.

Computed and measured stage comparisons for both the calibration and verification periods are shown for the four internal tide-gage sites in figure 17. Table 2 summarizes the standard errors for each day and lists the average standard error for calibration and verification periods. The average error for the verification period, 0.030 ft, is, not unexpectedly,

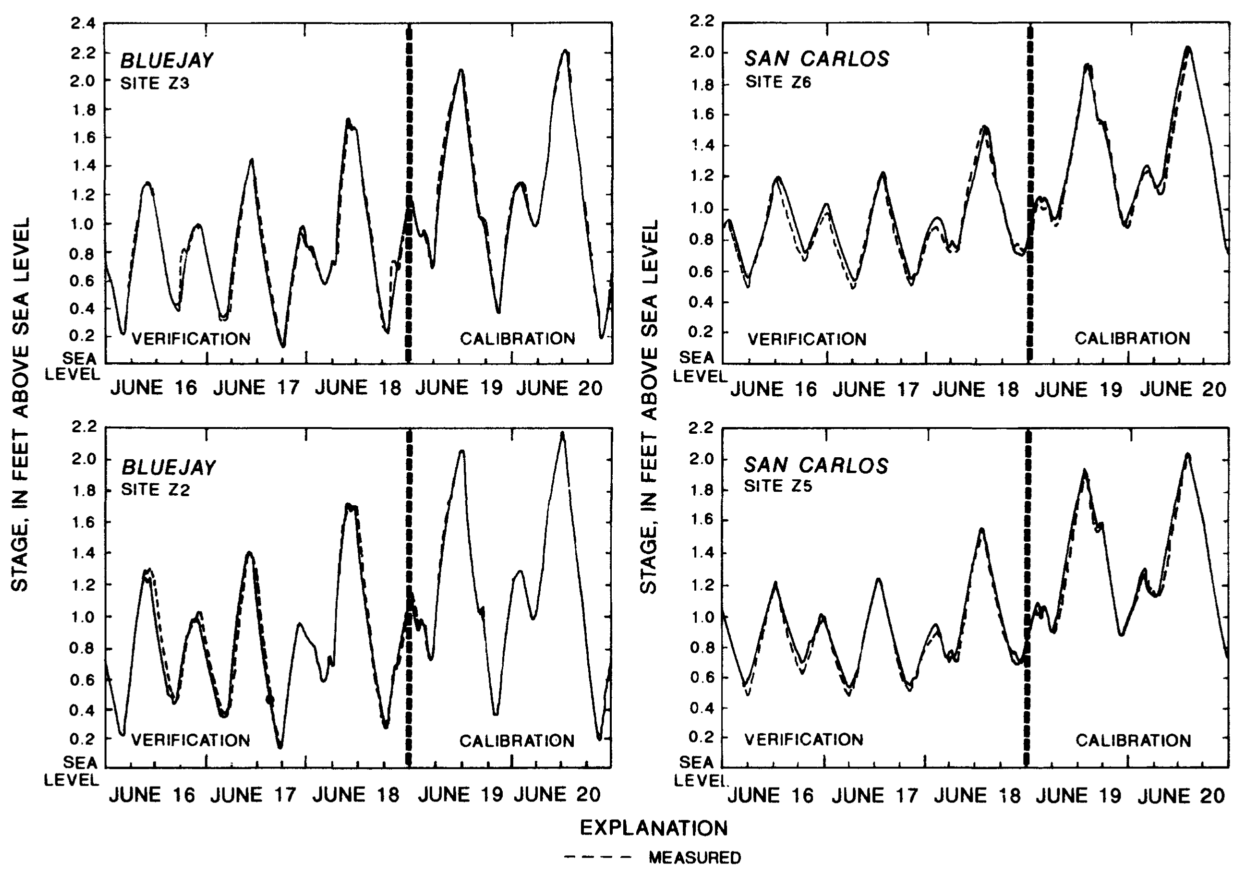


Figure 17. Measured and computed stage during calibration and verification periods at selected sites in the Bluejay and San Carlos canal systems.

Table 2. Standard error[1] and nominal standard error[2] for hydrodynamic model calibration (stage only) and verification (stage and discharge)
[ft³/s, cubic feet per second]

Calibration - stage				
	Standard error, in feet/nominal standard error for stage			
Date	Z2	Z3	Z4	Z5
6-19-86	0.023/0.014	0.013/0.008	0.027/0.025	0.020/0.019
6-20-86	0.017/0.008	0.017/0.008	0.033/0.025	0.018/0.014

Mean standard error = 0.026 foot
Mean nominal standard error = 0.015

Verification - stage				
	Standard error, in feet/nominal standard error for stage			
Date	Z2	Z3	Z4	Z5
6-16-86	0.030/0.028	0.036/0.034	0.033/0.043	0.032/0.046
6-17-86	0.020/0.016	0.032/0.025	0.036/0.048	0.030/0.040
6-18-86	0.026/0.018	0.016/0.011	0.038/0.044	0.030/0.034

Mean standard error = 0.030 foot
Mean nominal standard error = 0.032

Verification - discharge					
	Standard error, in ft³/s/nominal standard error for discharge				
Date	Q1	Q2	Q3	Q4	Q5
6-19-20-86	59/0.031	23/0.043	19/0.025	29/0.032	29/0.068

Mean standard error = 32 ft³/s
Mean nominal standard error = 0.040

[1]Standard error $= \frac{\Sigma \left| \text{computed} - \text{measured} \right|}{\text{number of observations}}$.

[2]Nominal standard error $= \frac{\text{standard error}}{\text{range of observations}}$, dimensionless.

slightly higher than for the calibration period. This error is about 3 percent of the average range in tidal stage in the study area and indicates excellent agreement between the model and the real canal systems.

Results of more sensitive model verification using computed and measured discharges at five sites (see fig. 7) are shown in figure 18. The standard error, also shown in table 2, is computed in a similar manner as described for stage. The standard error ranges from 19 to 59 ft³/s, which represents an average 4 percent of the range in discharge computed at each site. This degree of comparison is within the expected accuracy of the field-measured discharges and is a strong indication that the hydrodynamic model adequately simulates the real canal system for the purposes of this study.

One unexplained discrepancy between measured and computed discharges occurred at site Q5 (fig. 7) within the San Carlos system where model results frequently exceeded measured discharges during floodtide (water flow from east to west at this site). The most likely cause of this was a significantly nonstandard vertical velocity distribution during floodflow at the bridge that resulted in measured velocities lower than the model-computed vertical averages. To keep the time necessary to make individual discharge measurements as brief as possible with minimal sacrifice in accuracy, velocity readings were made at only one point, six-tenths of the total depth, in each transverse subsection. This method would not have detected unusually high velocities in the upper part of the 6-ft-deep measuring section.

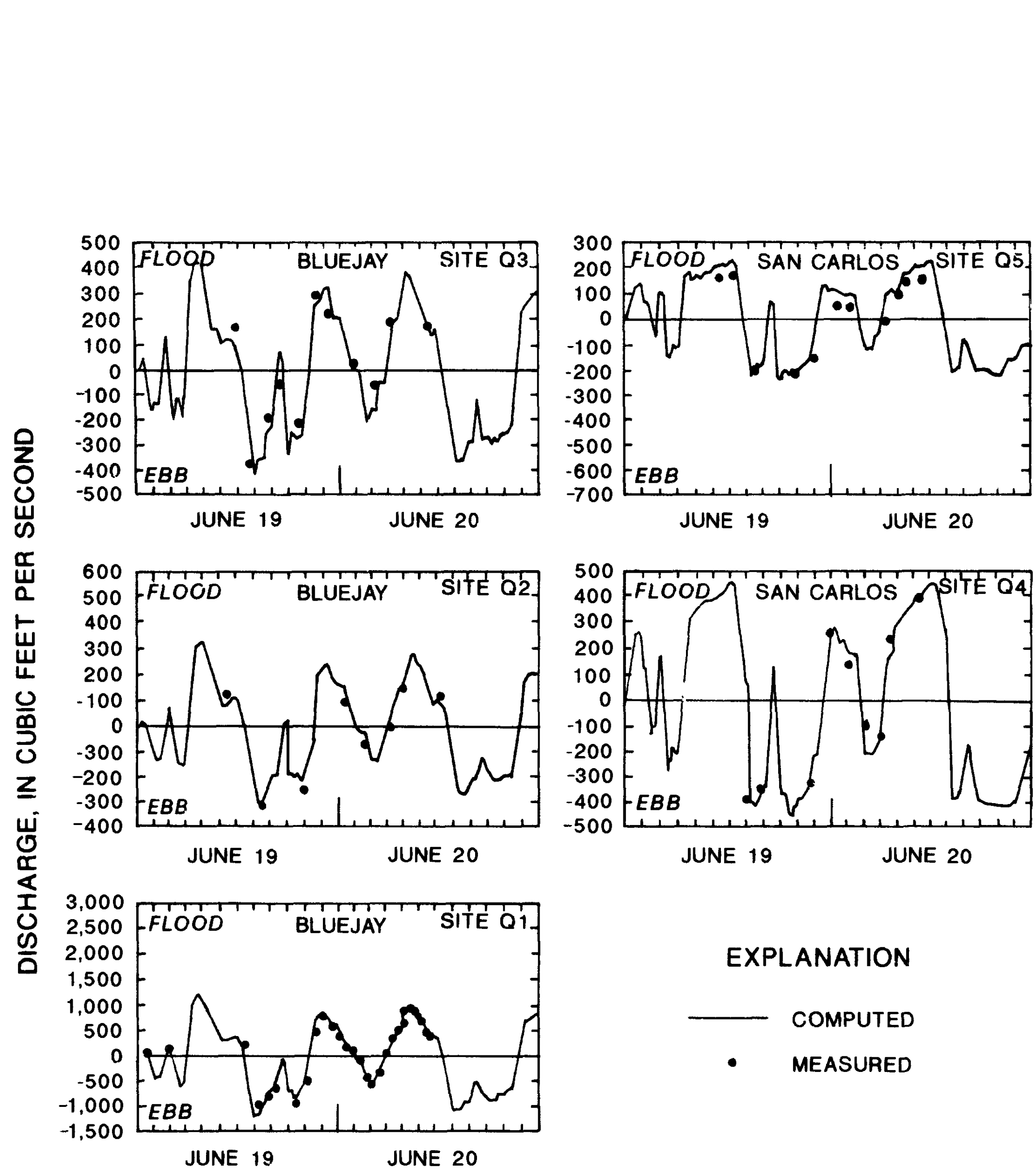


Figure 18. Measured and computed discharge during verification period at selected sites in the Bluejay and San Carlos canal systems.

Another interesting observation regarding computed versus measured discharges can be made for the period between 1600 and 1800 hours on June 19, 1986, when a thunderstorm affected the study area. Apparently, the combination of low barometric pressure and stormwater runoff caused the measured tidal stage throughout the Bluejay and the San Carlos canal systems to rise slightly in the midst of a normally rapidly falling phase of the tide (fig. 9). These conditions also were reflected in the model boundary value data and, therefore, affected the simulation results. The number of discharge measurements performed during the storm were limited due to safety considerations, but the few that were made had surprisingly good agreement with the computed results considering that density stratification effects were assumed negligible in model formulation and freshwater inflow was set to zero for the Cape Coral application. It is fortuitous that the gross features of stormwater influence on tidal flow in the canals seem to be well simulated by a branched 'one-dimensional model that is not designed to account for many physical aspects of the situation.

SIMULATION OF CIRCULATION AS AFFECTED BY TIDE GATES

After calibration and verification of an operational hydrodynamic simulation model for the existing Bluejay and San Carlos canal systems, modifications were made to simulate the operation of tide gates and to determine the effects of possible canal interconnections on tidal flow and circulation in the systems. Proposed installation of tide gates was limited to construction of new canal links between the Bluejay, San Carlos, and Rubicon canal systems (fig. 4). Because tide gates preclude navigation, possible installations in existing navigable waterways was not considered.

Tide Gate Description and Operation

As used in this report, tide gates are simple structures that function in exactly the same manner as a valve in a pump. Such valves provide a means to allow a fluid to flow in one direction and not the other. The principle is the same whether the valve is in a hand-operated tire pump, the human heart, or a tide gate. The basic features of a tide gate, including a top pivot that allows the gate to swing from its vertical or closed orientation when the upstream stage becomes greater than the downstream stage, are shown in figure 19.

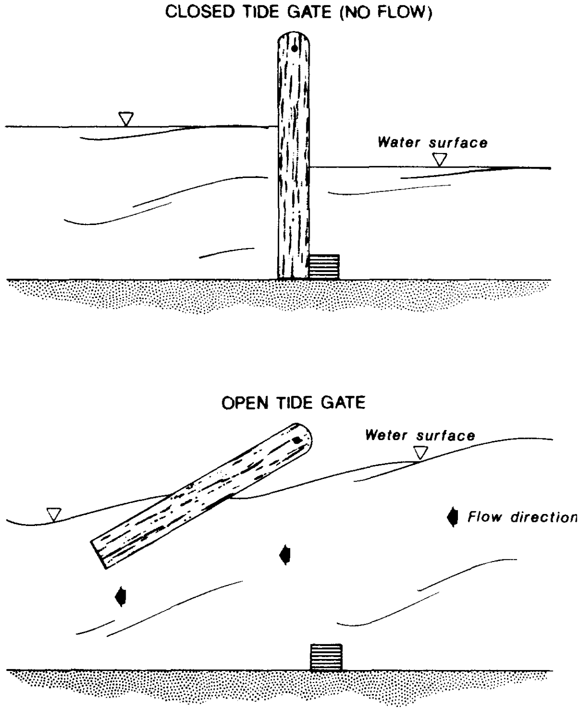


Figure 19. Schematic showing tide-gate operation.

Tide gates can operate with no external power supply other than the power inherent in the tide itself. This means that, once constructed, tide-gate operation is automatic and requires no attention other than periodic maintenance.

Tidal discharge in a hypothetical canal for conditions before and after construction of a tide gate is shown in figure 20. The tides on each end of the canal in both instances are the same. Without a tide gate, the flow in the canal responds directly to the water-level gradient imposed by the tide. The resulting discharge is the same in both directions, which cancels out when averaged over each tidal cycle. With a tide gate, the flow in the direction of the open gate is the same as with no gate. When the flow starts to reverse, however, the gate closes and the discharge in the canal is zero until the gate reopens. In this case, a net discharge or circulation in the direction of the open gate is produced.

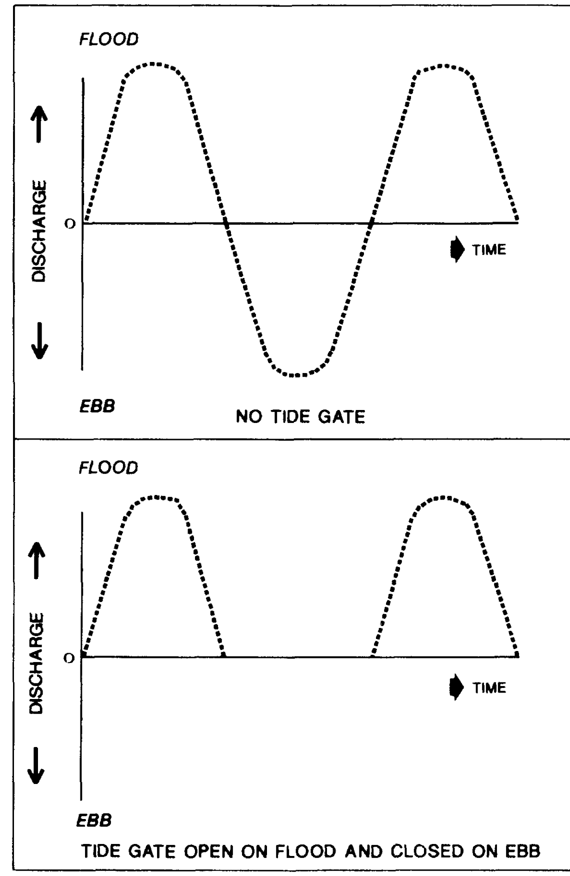


Figure 20. Hypothetical tidal discharge in canal with and without a tide gate.

This is a simplified example to demonstrate the concept of tide-gate operation and the means by which circulation may be induced in the study area. In reality, tides are complex, vary from day to day, and may not necessarily be the same before and after tide-gate construction. In a canal system such as Cape Coral, the action of a tide gate itself can have localized feedback effects on the tide within the canals due to necessary adjustments to a new distribution of water. These situations are automatically accounted for in a properly calibrated hydrodynamic model such as that being used in this study.

Modification of BRANCH Model for Tide Gate Simulation

The BRANCH model, as described by Schaffranek and others (1981), has no capability to simulate the action of tide gates. To meet the needs of this study, model modifications were made to add this capability at internal branch junctions connecting only two branches. Prior to modification, internal boundary conditions at a representative junction, k, connecting an upstream (subscript u) and downstream (subscript d) branch are defined as

$$Z_u = Z_d, \text{ and} \quad (3)$$

$$Q_u + Q_d = W_k \quad (4)$$

where

Z is water-surface elevation,

Q is discharge, and

W_k is a user-specified external flow at junction k that must be zero in conjunction with tide-gate simulation.

The first condition causes a continuous water surface at the junction, and the second condition enforces mass continuity.

An open tide gate at junction k is assumed to have no effect on the flow, so the internal boundary conditions in this circumstance also are represented by equations 3 and 4. A closed tide gate, however, creates two adjacent dead-end branches with unequal water levels that have the following boundary conditions.

$$Q_u = Q_d = 0. \quad (5)$$

In addition to the proper boundary conditions, tide-gate simulation also requires that the gates dynamically open and close in response to prevailing computed hydraulic conditions. An existing linear time extrapolation feature within BRANCH was changed to a parabolic time extrapolation and used to determine if flow at an open tide gate is predicted to reverse during the next time step. If so, the tide gate was closed by assigning the boundary condition expressed by equation 5 prior to the start of computations for the next time step.

In a similar fashion, parabolic time extrapolation was used in the case of a closed tide gate to determine whether the upstream stage is predicted to be greater than the downstream stage. If so, the gate was opened by assigning boundary conditions expressed by equations 3 and 4 prior to the start of computations for the next time step.

Because extrapolations are subject to error, it is possible that gate openings or closings based solely on extrapolated information also will be erroneous. To test for this possibility, an algorithm was inserted into the model at the end of each time step to assure that any gate changes were done appropriately. The same algorithm also checks for gate opening or closing conditions that were not detected due to extrapolation error. The gates also were allowed, at the user's choice, to open in either direction to provide complete flexibility in determining optimum placement and operation of proposed tide gates.

Optimization of Tide Gate Locations

Prior to performing simulations of possible modified canal systems, it was necessary to address two specific issues. The first issue concerns the number of possible gate settings (closed, completely open, open in one direction, open in the other direction) and possible gate combinations. With 11 possible gates and 4 potential settings, the total number of computer runs needed to evaluate all possible gate combinations would be nearly 4.2 million. The second issue was the determination of a suitable criterion for judging the relative effectiveness of the various gate combinations and settings.

Several concepts were used to reduce the number of computer runs. The first was based on a recognition that most of the hydraulic benefits to the study area from tide gates will probably be achieved well before activation of all 11 gates. Evaluation of the most effective single tide gate and the most effective combinations of two to four tide gates was undertaken in this study.

The second concept resulted from the observation, after making several initial runs, that the stage relation between the two canal systems in the study area establishes a preferred direction of water motion from the San Carlos system to the Bluejay system. It was possible, on this basis, to eliminate many gate combinations that would induce the reverse direction of flow and thereby significantly reduce the number of model runs.

A third pragmatic concept derives from the recognition that any tide-gate construction program must be progressive. For example, the chosen two-gate combination must include the gate already constructed as the best single gate, and so forth. Use of this concept also reduced the number of options that required testing. By application of these concepts, the total number of computer runs needed to meet the objectives of this study was reduced from a potential 4.2 million to 150, still a large, but manageable, number.

An appropriate measure of the ability of tide gates to improve circulation within the study area was needed to determine optimum tide-gate locations and combinations. Several such measures, called objective functions, were evaluated, and the one chosen for use in this study was

$$Q_v = \frac{\sum_{i=1}^{n} \left|Q_i\right| \cdot V_i}{\sum_{i=1}^{n} V_i},$$

where

Q_v is the tidally averaged, volume-weighted, discharge objective function,

Q_i is the tidally averaged discharge in segment i,

V_i is the volume of segment i at sea level, and

n is the number of segments in the model.

The primary reason for use of this objective function is its inherent emphasis on segments having large volumes.

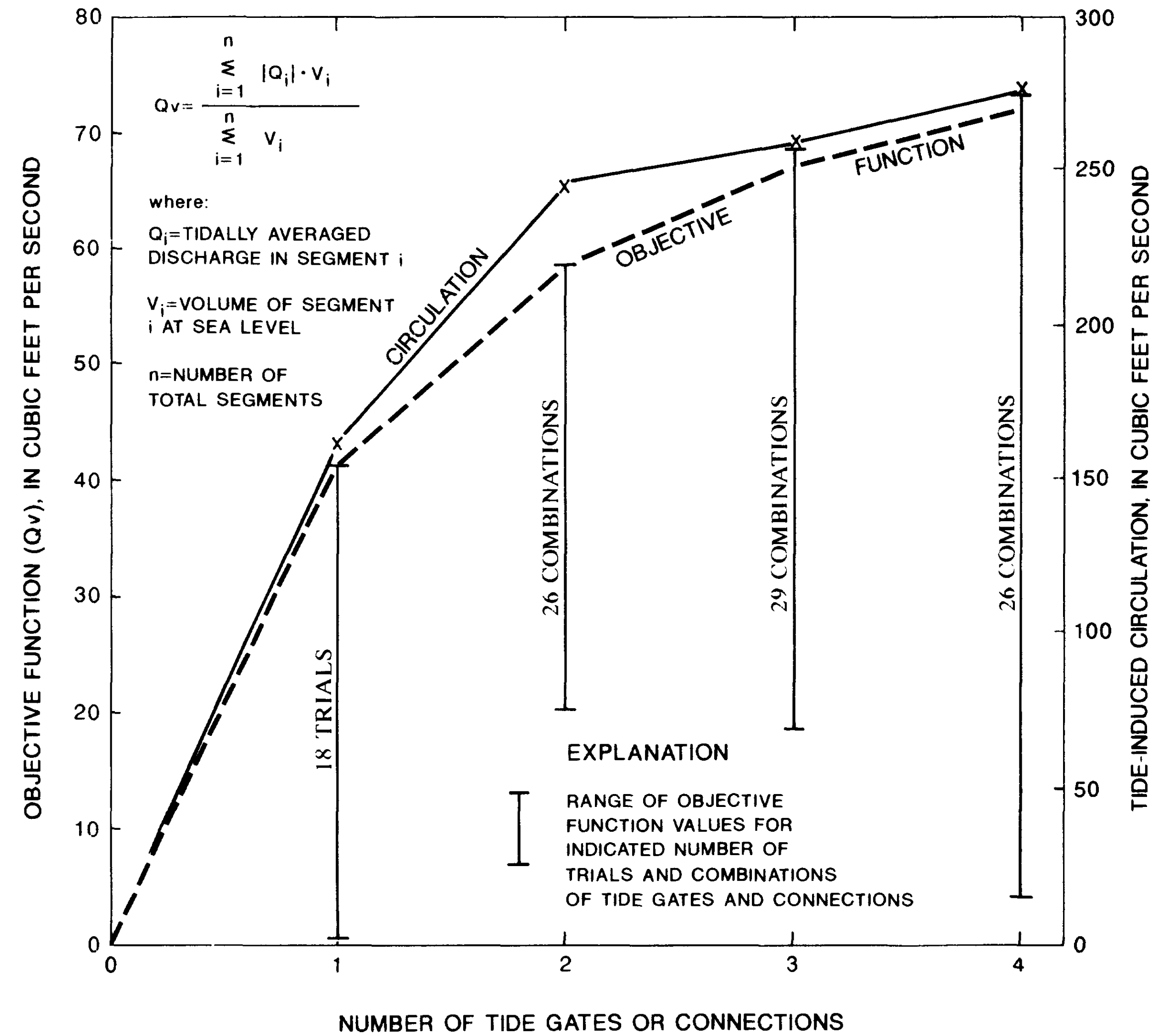


Figure 21. Effect of number of tide gates and interconnections on the objective function and circulation.

Operationally, this means that large induced circulation (tidally averaged discharge) in lakes (segments with large volumes) will produce large values of the objective function. These parts of the study area appear to have the most severe water-quality problems, as indicated in data from Douglas Morrison (City of Cape Coral, written commun., 1986) and data collected during this study.

Another variable of interest, although not used as the objective function, is the total induced circulation or tidally averaged discharge in the study area. This was operationally defined as the sum of tidally averaged segment discharges at all model boundaries that have outgoing flow and expressed as

$$Q_c = \sum_m |Q_j| ,$$

where

Q_c is induced circulation,

Q_j is tidally averaged discharge at boundary segment j, and

m is the set of boundary segment numbers for segments having outgoing, tidally averaged discharge.

Simulation Results for Modified Canal System

For meaningful comparison of improvements in tidal circulation of the study area using different tide-gate combinations, a repeating tide (one having the same stage at the beginning and end of a day) was necessary for accurate circulation computations. This eliminates the effect of differing

Table 3. Summary of tide-gate[1] analysis

[All values are in cubic feet per second]

Gate location (fig. 18)	Type of interconnection[2]	Circulation		Objective function	
		Added	Cumulative	Added	Cumulative
A	a	168	168	41.5	41.5
B	a	79	247	17.4	58.9
C	a	11	258	8.6	67.5
D	b	21	279	4.7	72.2

[1]All gates are assumed to be 10 feet wide and 5 feet deep. Other dimensions will alter the results presented in this table.

[2]a = tide gate with flow from east to west or north to south; b = no gate, open-channel flow.

amounts of water storage within the canal systems. A repeating 24-hour tidal-stage input was first generated for each boundary and was based on the spring tide that occurred on June 7, 1986. For comparison of circulation during neap tides, a second set of repeating boundary conditions was developed based on a tide that occurred on June 16, 1986.

Results of model simulations are summarized in figure 21, which shows the maximum value achieved for the objective function and the corresponding tide-induced circulation through the modeled area for the applicable number of tide gates or interconnections. Note that there is no circulation for the existing condition. Also shown is the range in objective function values and the number of computer runs made for each number of gates. This figure shows that the largest increase in the objective function occurs for the first gate and that additional gates result in successively diminishing returns. Four gates will produce less than two times the effectiveness of a single gate.

The order of gate construction (keyed to fig. 16), type of gate, amount of circulation and objective function value added by each successive gate, and cumulative values are summarized in table 3. Flow paths through the study area and circulation induced in each path, created by the addition of one to four tide gates, is shown in figures 22 through 25. The first gate (fig. 22) opened a single flow path entering the Bluejay system in the northeastern quadrant of Lake Thunderbird and induced a circulation of 168 ft^3/s everywhere along the path. Addition of the second gate (fig. 23) created a flow path into Lake Finisterre and increased the total circulation to 247 ft^3/s, which was nearly equally split between the two paths through Lake Thunderbird and Lake Finisterre. The third gate (fig. 24) connected Lake Thunderbird and Lake Britannia, both in the Bluejay system. The total circulation was slightly increased to 258 ft^3/s by this gate. The fourth connection (fig. 25) was between the San Carlos and the Rubicon systems and was not a tide gate but rather an unregulated canal interconnection. Very little eastward flow was computed at this site so a tide gate was considered unnecessary. This interconnection increased the total circulation to 279 ft^3/s.

The circulation depicted in figures 22 through 25 is based on tide-gate and nontide-gate interconnection widths of 10 ft and depths of 5 ft relative to sea level. Tide gates, of course, can be designed and constructed in various widths and depths, which presents a further opportunity to control the distribution of circulation within the study area. Figure 26 shows the nonlinear effect that changing the width of all tide gates simultaneously has on the total circulation and the objective function. A similar effect is likely to occur if the width were varied at each individual tide-gate site.

Based on this analysis, the optimum placement of one tide gate to maximize a volumetrically weighted objective function will produce an increase in circulation of 168 ft^3/s in the study area. Similar analysis of three tide gates and one noncontrolled interconnection shows an increase in circulation to 279 ft^3/s. At this rate, a volume of water equivalent to the total volume of water in the Bluejay and the San Carlos systems would be circulated in about 9 days. The results, therefore, indicate that tide gates can be effectively used to convert the potential available in tidal stage differences into kinetic energy sufficient to increase water velocities and induce water circulation in many parts of the study area.

SIMULATION OF TIDAL FLUSHING

To determine the probable effects of tide gates on the flushing of dissolved and suspended constituents in the study area, a one-dimensional, constituent-transport model was applied. The Branched Lagrangian Transport Model (BLTM), as described by Jobson and Schoellhamer (1987), was used because of its computative efficiency, numerical stability, accuracy, and capability to interface directly with BRANCH model output.

The BLTM model can simulate the transport and interaction of up to 10 water-quality constituents in a network of interconnected open channels that have unsteady and reversing flows. In order to operate, the model must be supplied with flow-field information and channel properties as a function of time at a sufficient number of fixed locations to adequately represent the variability found in the study area. The information needed includes discharge, cross-sectional area, and top width. For the Cape Coral application, output from the BRANCH model was used as the source of input to the BLTM so all the assumptions and limitations of the hydrodynamic computations are reflected in the transport computations.

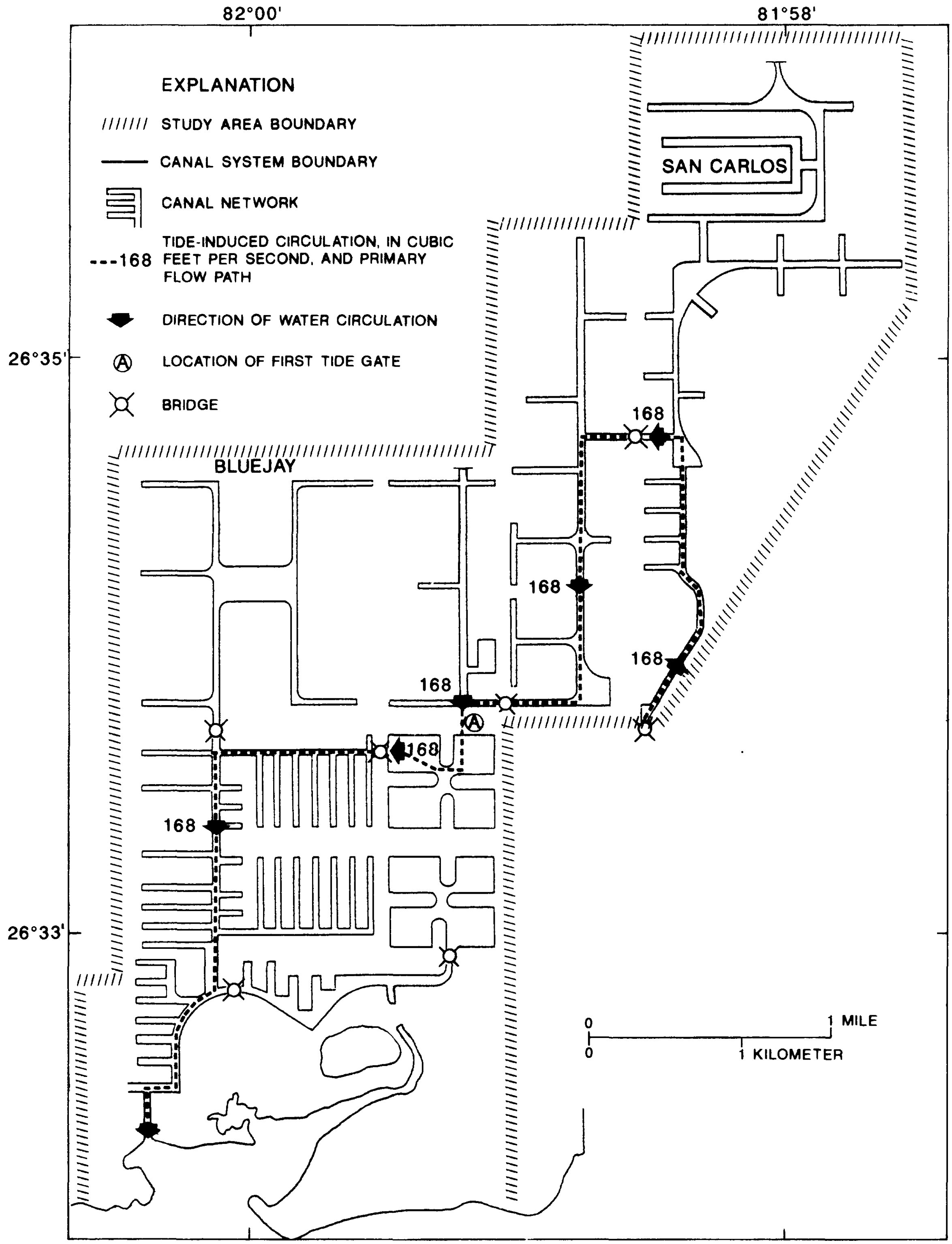


Figure 22. Flow path, circulation, and objective function for optimum single tide gate in the Bluejay and San Carlos canal systems.

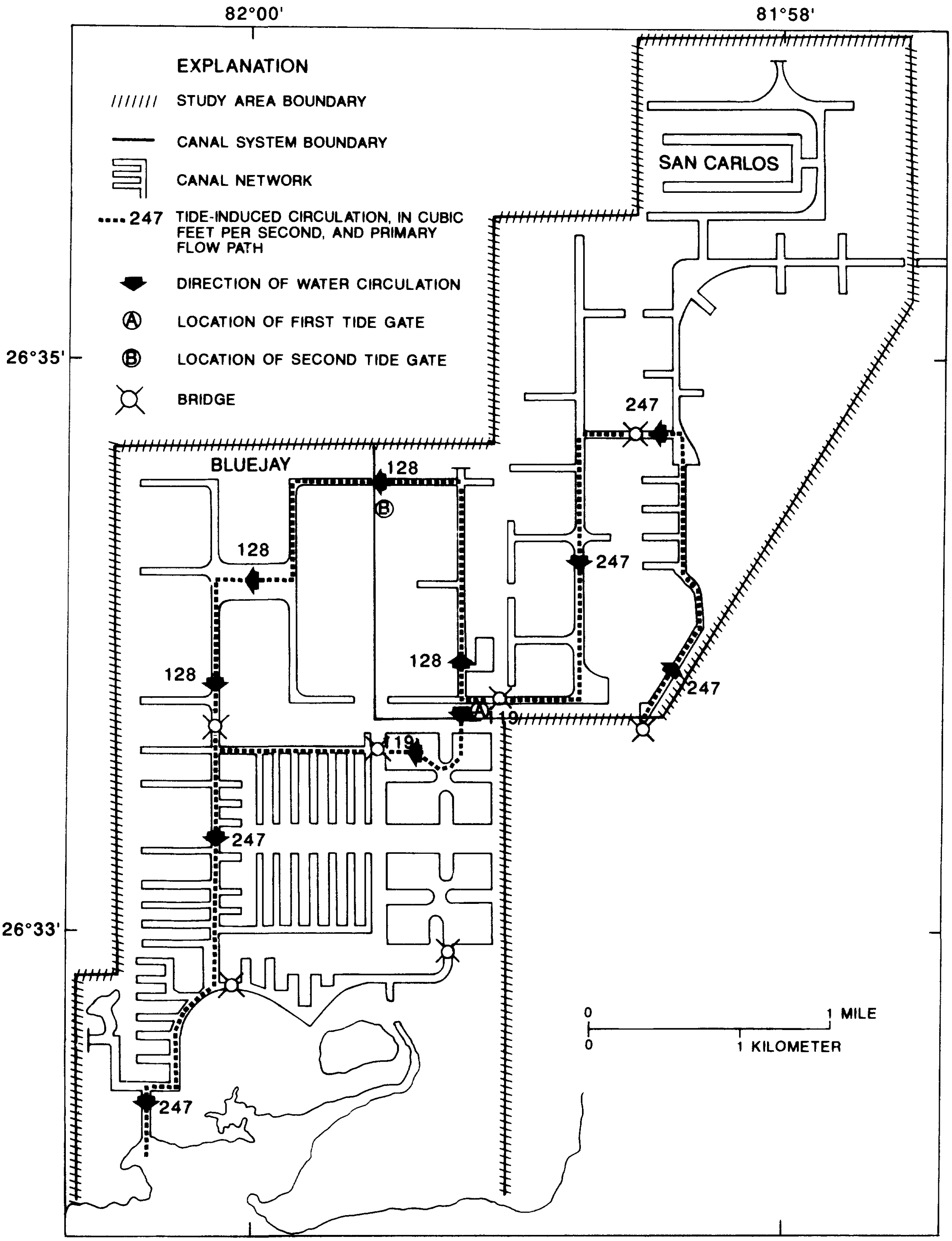


Figure 23. Flow path, circulation, and objective function for optimum two-gate combination in the Bluejay and San Carlos canal systems.

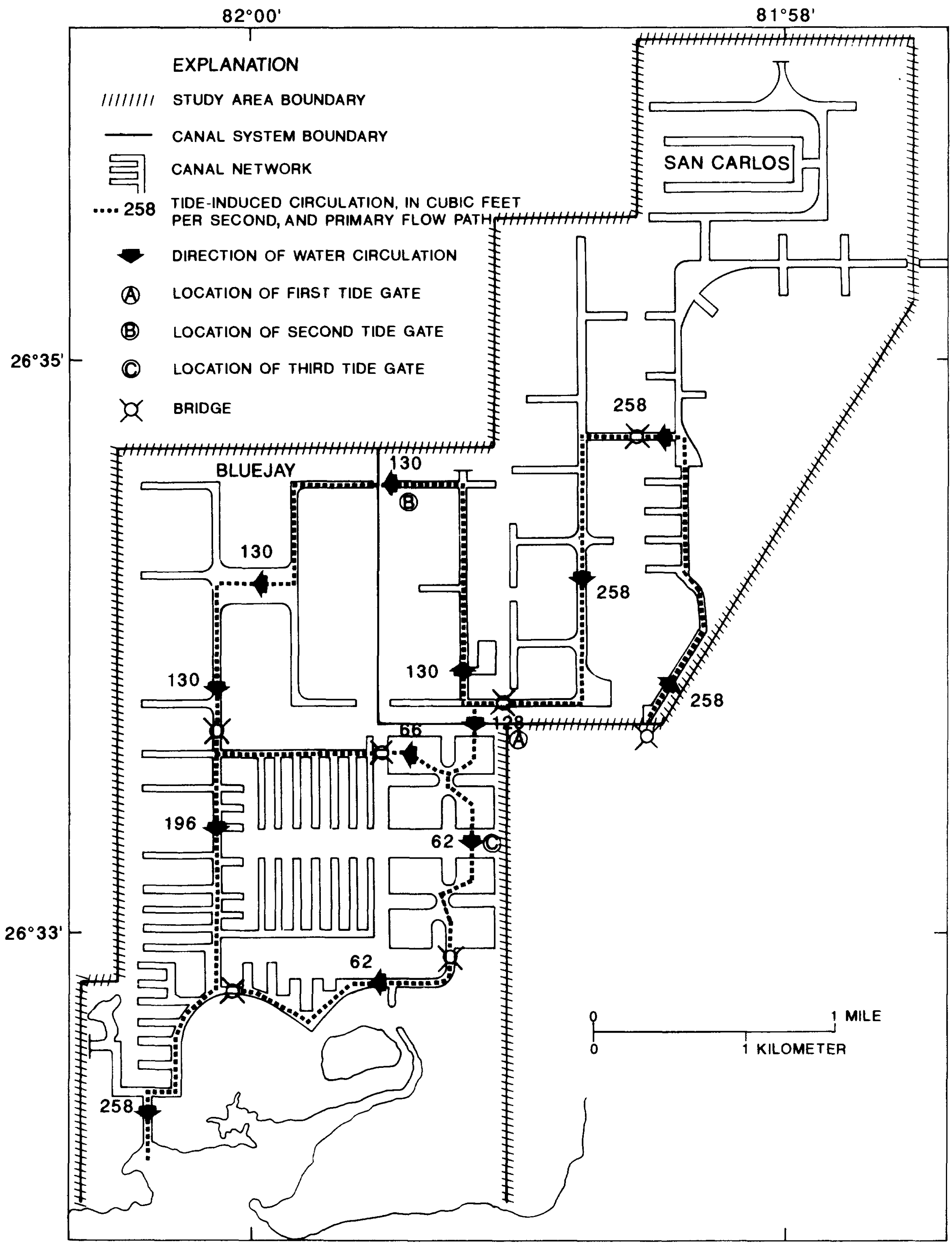


Figure 24. Flow path, circulation, and objective function for optimum three-gate combination in the Bluejay and San Carlos canal systems.

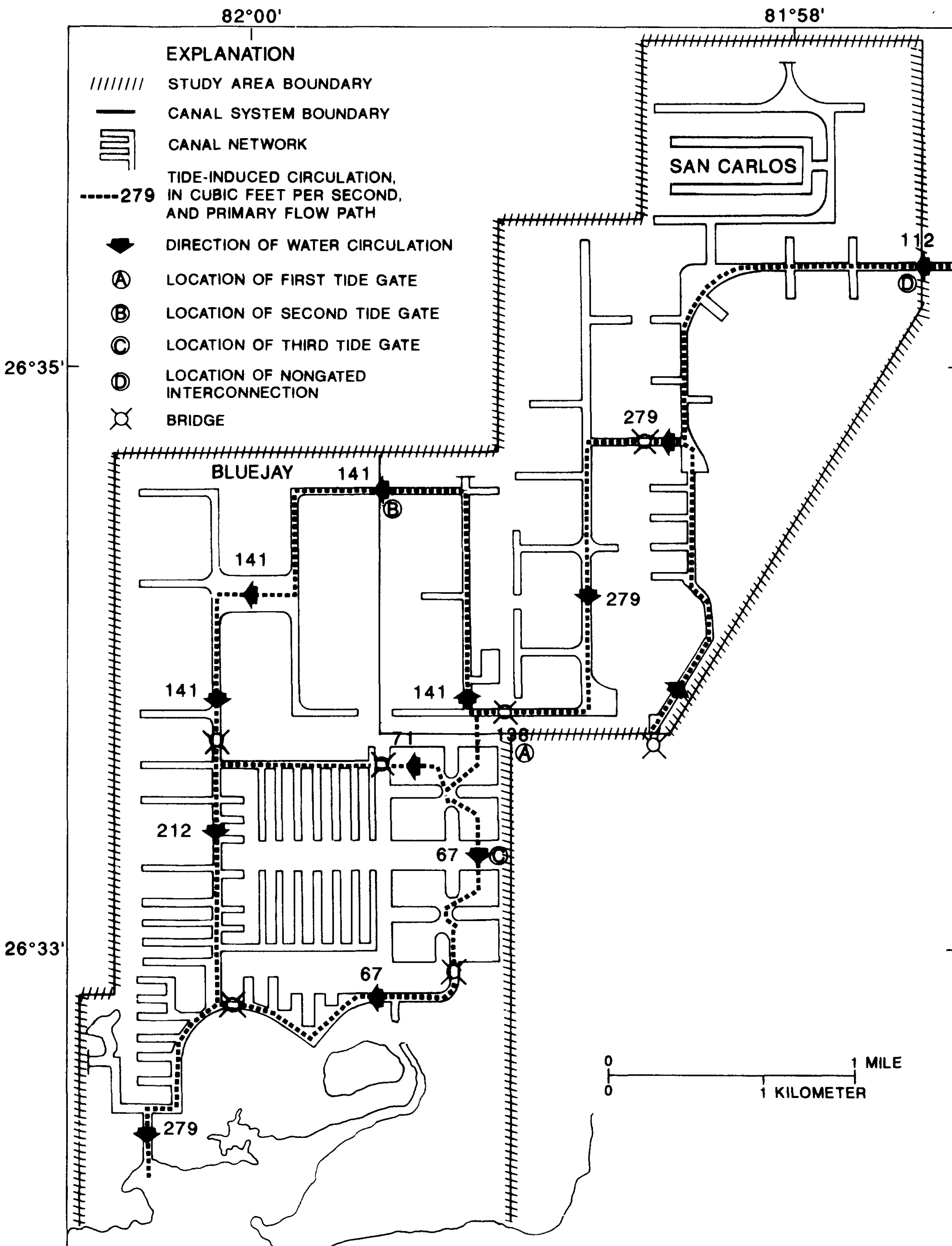


Figure 25. Flow path, circulation, and objective function for optimum four-gate combination in the Bluejay and San Carlos canal systems.

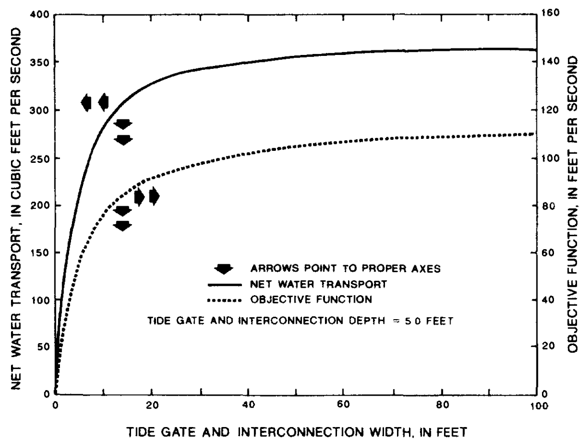


Figure 26. Effect of tide-gate width on circulation and objective function.

Governing Equations and Assumptions

The partial-differential equation for one-dimensional transport of a conservative substance, written in the Eulerian (fixed) reference frame, as given by Schoellhamer and Jobson (1986a), is

$$\frac{\partial C}{\partial t} = -U\frac{\partial C}{\partial x} + \frac{\partial}{\partial x}\left(D_x\frac{\partial C}{\partial x}\right) \qquad (6)$$

where

- C is the cross-sectionally averaged constituent concentration,
- U is the cross-sectionally averaged velocity,
- D_x is the longitudinal dispersion coefficient,
- x is longitudinal distance, and
- t is time.

This equation is often called the advection-dispersion equation because the first term on the right side of the equation accounts for the advection or movement of constituent mass due to average canal or river flow, and the second term accounts for constituent dispersion that occurs primarily because all of the water does not move at the average velocity. The second term has been characterized as a correction factor needed to account for the fact that a three-dimensional problem has been reduced to a one-dimensional equation (Jobson, 1980).

The advective term has caused many problems in development of numerical solutions to equation 6, including numerical dispersion, undershoot, overshoot, negative concentrations, oscillations, and instabilities (Schoellhamer and Jobson, 1986a). To minimize these problems, Fischer (1972) used a Lagrangian reference frame to simulate constituent transport in Bolinas Lagoon, Calif. Conceptually, this is a reference frame that follows a series of completely mixed, individual fluid parcels while keeping track of all the factors that change the concentration within each parcel. Because the reference point moves at the same velocity as the water, U in the advective term of equation 6 becomes zero, and the one-dimensional, Lagrangian transport equation for a conservative substance, as given by Schoellhamer and Jobson (1986a), becomes

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial \xi}\left(D_x\frac{\partial C}{\partial \xi}\right) \qquad (7)$$

in which the Eulerian distance coordinate x is determined at any time by its relation to the Lagrangian distance coordinate ξ, as follows

$$\xi = x - x_o - \int_{t_o}^{t} u\,dt \qquad (8)$$

where x_o is the Eulerian location of a water parcel at time t_o.

Use of equation 8, which represents the key concept of the Lagrangian modeling technique, requires recognition that the distance (ξ) a water parcel moves from its Lagrangian reference point is always zero because the frame of reference moves with the parcel. Setting $\xi = 0$ allows the Eulerian distance (x) that a parcel travels from a fixed reference point to be determined. Under constant velocity conditions, for example, the change in parcel position, Δx, over a time period, Δt, is simply $U\Delta t$.

Elimination of the advective term and a change in how the longitudinal distance coordinate is computed are the only differences between equations 6 and 7 caused by adoption of the Lagrangian reference frame. The primary gain is the simplicity, stability, and accuracy afforded by not having to numerically formulate the advective term. The cost for these benefits is a greater bookkeeping effort to track the position of all the water parcels continuously. Because computers are well suited for bookkeeping activities, the advantages of the Lagrangian reference frame outweigh the disadvantages (Jobson, 1980).

Although this study is limited to evaluation of the transport of conservative substances, the BLTM model is capable of simulating nonconservative and multiconstituent transport using either QUAL-II (Roesner and others, 1977a; 1977b) or other user supplied reaction kinetics by numerical solution of

$$\frac{\partial C_\ell}{\partial t} = \frac{\partial}{\partial \xi}\left(D\frac{\partial C_\ell}{\partial \xi}\right) + S_\ell + \Phi_\ell + \sum_{n=1}^{m} K_{\ell,n}\left(C_n - CR_{\ell,n}\right) \qquad (9)$$

in which C_ℓ, S_ℓ, and Φ_ℓ are concentration, zero-order rate of production of concentration, and rate of change of concentration due to tributary inflow for constituent ℓ, respectively. The summation allows constituent interaction where $K_{\ell,n}$ is the rate coefficient for the production of constituent ℓ due to the presence of constituent n at concentration C_n, and $CR_{\ell,n}$ is the concentration of constituent n at which the production of constituent ℓ due to constituent n ceases. Full explanations of these capabilities are given by Jobson (1980), Schoellhamer and Jobson (1986a; 1986b), and Jobson and Schoellhamer (1987). Jobson (1985) reports successful application of this technique to the Chattahoochee River downstream from Atlanta, Ga.

Numerical Methods

Constituent advection in the BLTM is handled by moving water parcels and their associated constituents according to the hydraulic and physical properties of the modeled system that are supplied as input. The solution starts with a series of initial parcels, defined by the user, initial concentration values in each parcel, and concentration values at each location where water flows into the modeled system. Generally, one or more parcels are defined within each branch of the system. As new water enters the system during a simulation, parcels are added that have a volume equal to the inflow volume during each time step.

As advection occurs in real rivers and channels, some water moves faster and some moves slower than average. This is depicted schematically by the velocity profiles in figure 27. Water near the bottom and sides of channels generally moves slower than water near the surface and middle. As a result, a parcel, K, moving with the average stream velocity must contribute some volume of water to and receive an equivalent volume from its adjacent downstream parcel, K+1 (fig. 27). The same process, of course, applies at the boundary with the upstream parcel, K-1. This interparcel flow also is accompanied by a transfer of constituent mass between parcels. The net effect is that parcels that have higher constituent concentrations than their neighbors lose constituent mass and vice versa. In this manner, dissolved and suspended material is spread or dispersed longitudinally.

The dispersion concept used in the BLTM is simple, yet effective. It is assumed that the amount of water exchanged (DQ) at each parcel boundary is a user-defined fraction (D_f) of the total channel discharge (Q) at the same location,

$$D_Q = D_f \cdot Q. \tag{10}$$

Using simple finite-differences, Schoellhamer and Jobson (1986b) showed that D_f can also be expressed as

$$D_f = \frac{D_x}{U \cdot \Delta x} = \frac{1}{P} \tag{11}$$

in which P is the Peclet number, a dimensionless ratio of advective and dispersive flow properties, a strong indication that the assumed definition of D_f (eq. 10) is valid.

Schoellhamer and Jobson (1986b) concluded that D_f values in the range of 0.1 to 0.4 provide the greatest model stability and accuracy. They also show that the determination of D_f for use in the model is both a function of the longitudinal dispersion coefficient, D_x, as well as the model time step by substituting the steady-flow expression

$$\Delta x = U \cdot \Delta t \tag{12}$$

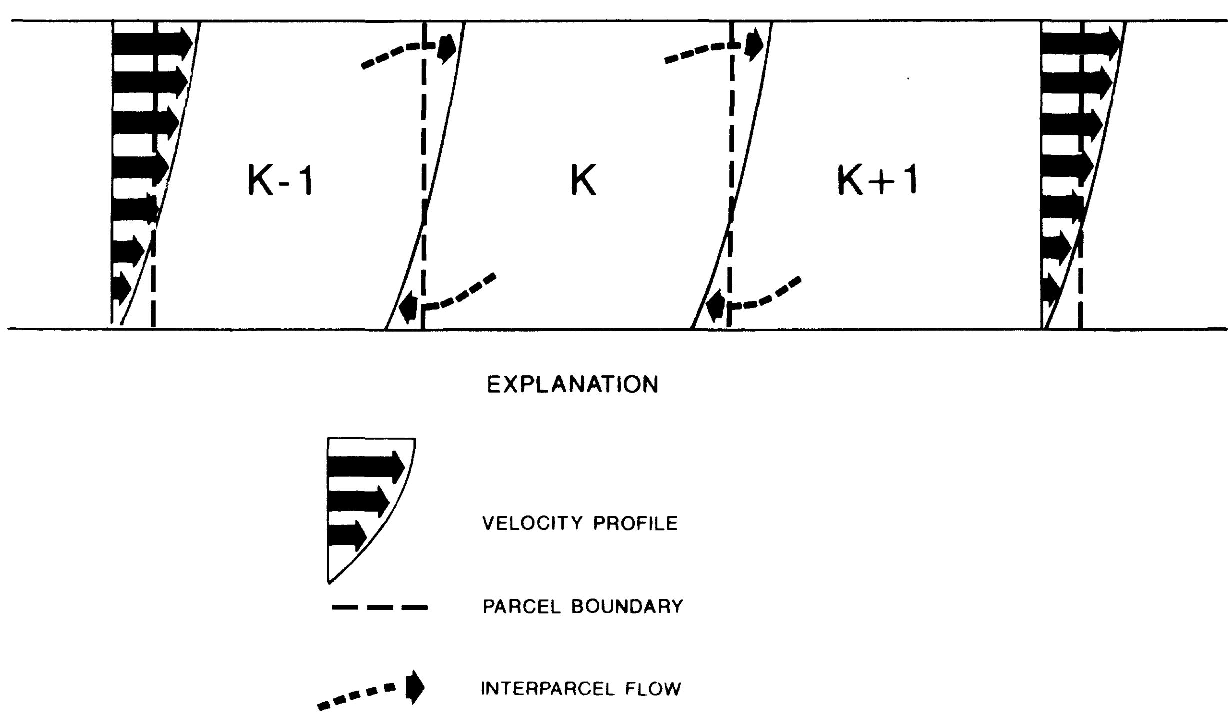


Figure 27. Schematic showing interparcel flows.

into equation 11, resulting in

$$D_f = \frac{D_x}{U^2 \Delta t}. \qquad (13)$$

This means that the user can assure a value of D_f in the optimal range by knowing the longitudinal dispersion coefficient, by choosing a representative tidal velocity, and by properly selecting the model time step.

Selection of Model Input Parameters

The selection of a single time step that would allow accurate simulation of dispersion in all parts of the study area was not possible because of the large range of representative tidal velocities in the various canals. Near dead ends and in lakes, tidal velocities of less than 0.01 ft/s were common. In other canals, tidal velocities averaging more than 1.0 ft/s were measured during this study. With the longitudinal dispersion coefficients expected to vary less than one order of magnitude (U.S. Environmental Protection Agency, 1975), application of equation 13 would produce a possible range in D_f of at least two orders of magnitude.

To resolve this dilemma, the canals of the study area were conceptually divided into three categories on the basis of expected importance of dispersive processes. The first category includes canals with tidal velocities sufficiently large that advective processes are expected to dominate. The second category consists of dead-end canals in which tidal velocity is so low that advection is extremely small and dispersion is the only effective means of constituent transport. The third category includes canals or canal segments in which advection and dispersion are important mechanisms; these canals represent the transition or a link between dead-end canals and predominately advective canals.

It was considered most important for purposes of this study to properly simulate the dispersive processes in category 3 transition canals because they form a link between two extremes that can be reasonably approximated in other ways. The most important process in category 1 advective canals is easily handled because the BLTM accurately simulates advection. Dead-end, category 2 dispersive canals can be simulated using one or two completely mixed segments, recognizing that advective transport is inconsequential but also probably somewhat overstating the degree of dispersive transport.

The computation to determine the model time step was based on category 3 transition canals and uses a dispersion coefficient (D_x) of 1.0 ft^2/s from data presented by the U.S. Environmental Protection Agency (1975) and a velocity of 0.05 ft/s that typically occurs near the mouths of many of the dead-end canals in the study area. Application of equation 13 to these values produces a time step of 16.6 minutes with D_f = 0.4. The actual time step used in the model was 15 minutes, an integer multiple of the 5-minute time step used in BRANCH to compute the hydrodynamics.

The interparcel exchange coefficient (D_f) was uniformly applied to all model segments with the knowledge that the influence of this coefficient would be minimal in category 1 and 2 canals. The time step was chosen to provide for proper dispersive simulation in category 3 canals as well as optimum overall model stability and accuracy. Model sensitivity to this coefficient is within the expected uncertainty associated with other model assumptions; therefore, chosen D_f values are used with confidence. One area of uncertainty is dispersion computations within the lakes of the study area. This is not considered to be as well represented as in the canals because of the inherent limitations of a one-dimensional concept to describe two- and three-dimensional processes. Determining advective and dispersive processes in the lakes and their effect on the total system will require additional study.

Results of Model Applications

The BLTM model was applied to the Bluejay and the San Carlos canal systems to determine the flushing rate of a conservative substance under existing conditions and after installation of a series of from one to four tide gates. Tide-gate locations are presented in figures 22 through 25. The effect of the dispersion factor, D_f, on computed flushing also was investigated.

The method used to evaluate flushing efficiency is straightforward. All water within the modeled canal system was arbitrarily assigned an initial constituent concentration of 1, and all water entering the system through any boundary was assigned a concentration of 0. The total constituent mass retained within the system was computed at every model time step from which the amount leaving could be determined.

The stipulation of zero concentration at the boundaries assumed that all of the material leaving the system never reentered. This is more true of conditions following tide-gate installation because riverlike flow would introduce new water at the upstream boundary and discharge canal water at the downstream boundary. In the absence of tide gates, however, water discharged from the canals during ebbflow would mix with water outside the boundaries, and a fraction would undoubtedly reenter the system during the next or succeeding floodtides. Under this scenario, the evaluation method is expected to overestimate flushing in direct proportion to the fraction of canal water that reenters during floodflow.

The differences in flushing efficiencies between the selected system configurations are illustrated by figure 28, which shows the fraction of original mass remaining in the system over a period of 7 days. The flat parts of the curves indicate that, during floodtide, the mass of constituent within the system remains constant because the added water has an assigned concentration of zero. The rate of mass loss is largest during the first several days and progressively decreases with time. The rate of mass loss also increases with the number of tide gates. The results indicate that constituent flushing is dominated by the advective process. Changes in

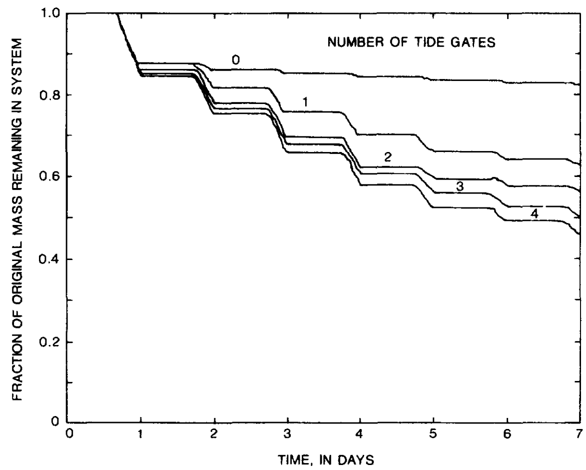


Figure 28. Short-term flushing due to tide gates.

the dispersion factor, Df, were observed to have little effect during the short, 7-day test.

The fraction of mass remaining in the system at the end of each day over a period of 50 days is shown in figure 29. The initial rapid mass loss due to advection induced by tide gates is easily distinguishable from the slower mass loss associated with dispersion. Also shown in figure 29 is the effect of halving the dispersion factor, which results in about 7 percent more canal water remaining in the system at the end of 50 days, but has a negligible effect over time periods of less than 10 days.

At the end of 50 days, the computed loss in constituent mass within the study area was about 27 percent for the existing condition with no tide gates, 53 percent with one tide gate, 61 percent with two tide gates, 74 percent with three tide gates, and 79 percent with four tide gates. This demonstrates that one or four tide gates can increase flushing in the study area from two to nearly three times, respectively. The results presented here can be considered a suboptimum condition based on tide gates of constant width. Greater flushing is likely if a concerted effort is made to properly size the gates.

The results from the BLTM in terms of constituent flushing from the system as a whole are shown in figures 28 and 29. Primary flow paths that are associated with each simulated tide gate and the locations of computed mass-reduction coefficients (F) are shown on a map of the study area in figure 30 and are listed in table 4. The mass-reduction coefficients in table 4 represent the fraction of original constituent mass (or concentration) computed to remain at each site after 50 days of model operation. This presentation allows detailed evaluation of many parts of the canal system. Some parts are flushed rapidly and others are flushed slowly. The primary flow paths created by the tide gates also are highlighted in figure 30 and are referenced in table 4.

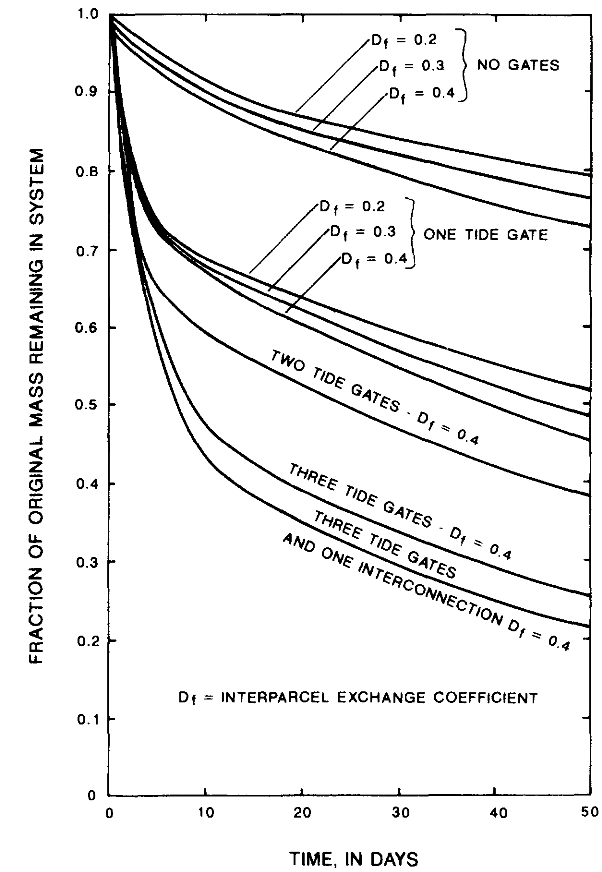


Figure 29. Long-term flushing due to tide gates.

WATER-QUALITY CHANGES DUE TO INCREASED CIRCULATION AND FLUSHING

Reliable predictions of changes that may occur in the concentrations of nonconservative water-quality constituents, such as dissolved oxygen, nutrients, suspended sediment, and phytoplankton, within the Bluejay and the San Carlos canal systems due to construction of tide gates are not possible using the results of this study alone. Accurate predictions might be possible by coupling the hydrodynamic and constituent-transport models developed by this study with reliable information on a variety of rate processes, such as sediment resuspension and settling, sediment oxygen demand, reaeration, photosynthesis, respiration, sediment nutrient exchange, denitrification, and stormwater loading. Until sufficient information is gathered to define these and other processes for incorporation into a nonconservative transport model, the probable water-quality changes that may result from tide-gate construction can be estimated only on the basis of the available information.

For conservative constituents, the model results presented in figure 30 and table 4 in terms of fraction of mass

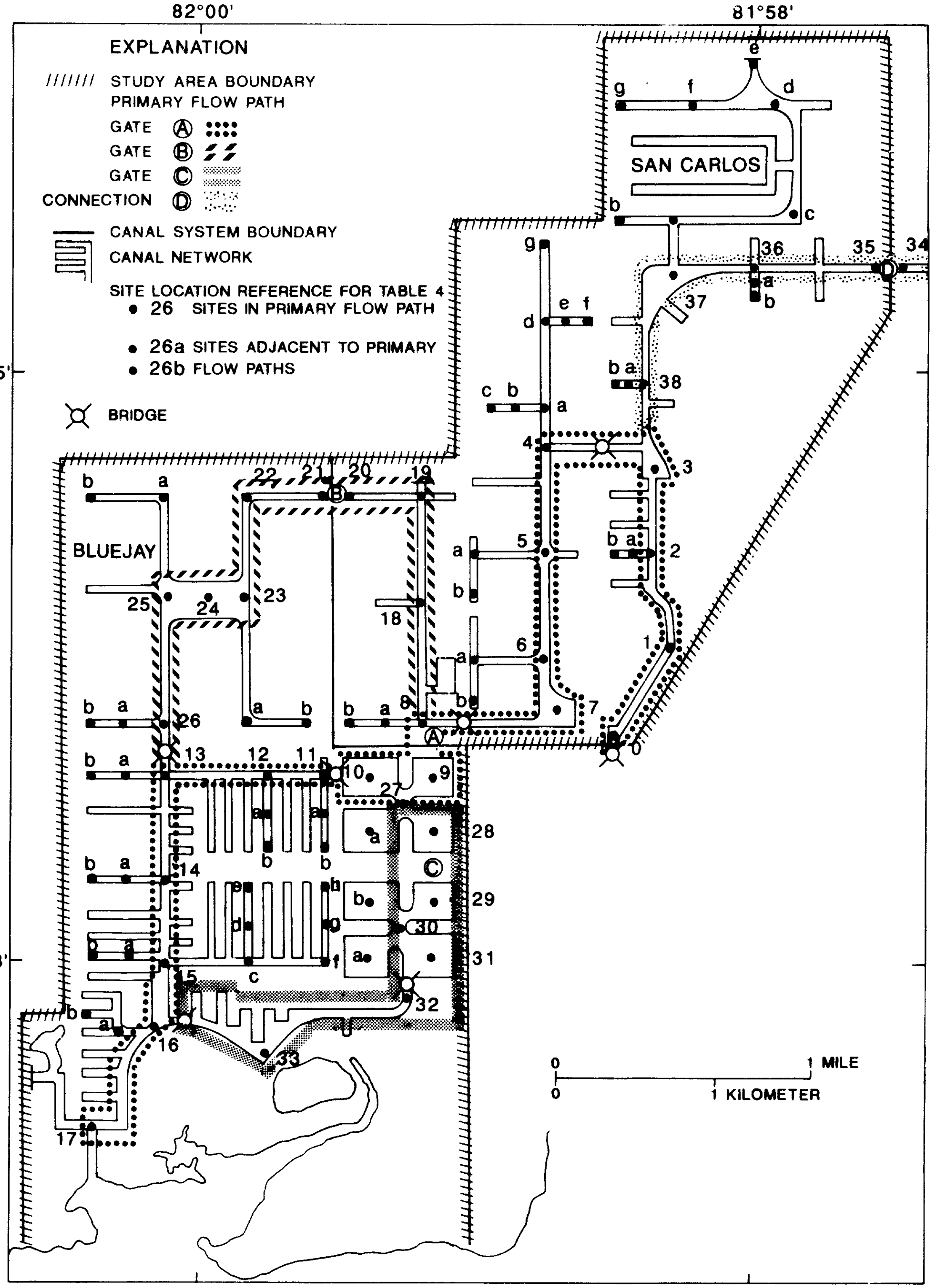


Figure 30. Flow paths caused by tide gates and locations of tabulated mass-reduction coefficients in the Bluejay and San Carlos canal systems.

Table 4. Constituent mass-reduction coefficients (F) computed for no tide gates and four tide-gate combinations

Flow path (see fig. 30	Site (see fig. 30)	Constitutent mass-reduction coefficients for indicated combinations of tide gates and interconnecting canal				
		None	A (fig. 23)	A, B (fig. 23)	A, B, C (fig. 24)	A, B, C, D (fig. 25)
A	0	0.06	0.00	0.00	0.00	0.00
A	1	.13	.00	.00	.00	.00
A	2	.45	.00	.00	.00	.00
A	2a	.77	.60	.58	.55	.60
A	2b	.97	.93	.94	.94	.94
A	3	.56	.01	.01	.01	.00
A	4	.78	.04	.02	.02	.00
A	4a	.95	.69	.65	.64	.65
A	4b	.99	.88	.87	.85	.87
A	4c	1.00	.98	.98	.98	.98
A	4d	1.00	.95	.93	.93	.95
A	4e	1.00	.97	.96	.96	.97
A	4f	1.00	.98	.98	.98	.98
A	4g	1.00	.98	.98	.98	.98
A	5	.94	.01	.02	.02	.01
A	5a	.98	.61	.62	.61	.62
A	5b	.99	.95	.95	.95	.95
A	6	.98	.02	.02	.02	.02
A	6a	.99	.63	.60	.61	.61
A	6b	1.00	.95	.95	.95	.95
A	7	.98	.00	.00	.00	.01
A	8	1.00	.01	.00	.00	.00
A	8a	1.00	.66	.63	.66	.64
A	8b	1.00	.94	.93	.94	.94
A	9	.96	.02	.01	.01	.00
A	10	.57	.02	.02	.01	.01
A	11	.50	.02	.02	.02	.01
A	11a	.86	.70	.62	.66	.69
A	11b	.98	.94	.94	.94	.93
A	12	.43	.02	.03	.03	.03
A	12a	.79	.58	.58	.59	.58
A	12b	.98	.94	.94	.94	.94
A	13	.29	.04	.02	.02	.02
A	13a	.72	.65	.60	.57	.59
A	13b	.98	.94	.94	.94	.94
A	14	.20	.05	.03	.02	.02
A	14a	.70	.59	.59	.57	.57
A	14b	.98	.94	.94	.94	.94
A	15	.13	.05	.03	.02	.02
A	15a	.67	.56	.55	.55	.57
A	15b	0.98	0.94	0.94	0.94	0.94
A	15c	.52	.45	.44	.48	.49
A	15d	.82	.79	.78	.84	.85
A	15e	.98	.96	.96	.97	.97
A	15f	.95	.94	.94	.95	.95
A	15g	.99	.99	.99	.99	.99
A	15h	1.00	1.00	1.00	1.00	1.00
A	16	.05	.06	.04	.03	.03
A	16a	.64	.58	.57	.55	.55
A	16b	.98	.94	.94	.94	.94

Table 4. Constituent mass-reduction coefficients (F) computed for no tide gates and four tide-gate combinations—Continued.

Flow path (see fig. 30	Site (see fig. 30)	Constitutent mass-reduction coefficients for indicated combinations of tide gates and interconnecting canal				
		None	A (fig. 23)	A, B (fig. 23)	A, B, C (fig. 24)	A, B, C, D (fig. 25)
A	17	.00	.00	.00	.00	.00
B	18	1.00	.76	.01	.01	.01
B	19	1.00	.97	.01	.01	.01
B	20	1.00	1.00	.02	.02	.01
B	21	1.00	1.00	.02	.02	.01
B	22	.95	.94	.01	.01	.01
B	23	.89	.85	.01	.01	.01
B	23a	.95	.94	.59	.60	.60
B	23b	1.00	1.00	.93	.94	.94
B	24	.74	.59	.01	.01	.01
B	25	.64	.50	.01	.01	.01
B	25a	.88	.82	.59	.58	.57
B	25b	.99	.96	.94	.94	.94
B	26	.38	.12	.02	.02	.01
B	26a	.76	.64	.60	.60	.57
B	26b	.98	.94	.94	.94	.94
C	27	.86	.01	.01	.02	.01
C	27a	.96	.42	.41	.37	.37
C	28	.97	.42	.40	.01	.01
C	29	.97	.97	.98	.02	.02
C	30	.88	.88	.87	.05	.03
C	30a	.97	.97	.97	.43	.44
C	30b	.97	.97	.97	.44	.46
C	31	.66	.65	.63	.04	.04
C	32	.62	.61	.60	.04	.04
C	33	.24	.20	.17	.04	.04
D	34	.00	.00	.00	.00	.00
D	35	1.00	1.00	1.00	1.00	.00
D	36	1.00	.98	.98	.97	.00
D	36a	1.00	1.00	1.00	.99	.61
D	36b	1.00	1.00	1.00	1.00	0.94
D	37	.97	.77	.75	.75	.00
D	37a	1.00	.95	.95	.95	.65
D	37b	1.00	.99	.99	.99	.96
D	37c	1.00	.95	.95	.95	.95
D	37d	1.00	1.00	1.00	1.00	.94
D	37e	1.00	1.00	1.0	1.00	1.00
D	37f	1.00	1.00	1.00	1.00	.98
D	37g	1.00	1.00	1.00	1.00	1.00
D	38	.80	.28	.26	.27	.00
D	38a	.95	.71	.73	.70	.61
D	38b	.98	.94	.94	.94	.92

remaining, F, can be used directly to estimate constituent concentration at selected locations or for the study area as a whole after tide-gate construction by applying the equation

$$Ce = Cr + F\,(Co - Cr) \qquad (14)$$

where

Ce is the estimated concentration at a site after tide-gate operation,

Co is the pregate concentration at a site, and

Cr is the concentration at the model boundary in the Caloosahatchee River.

When using equation 14, as long as model assumptions are met following tide-gate installation, water quality within the canal system with respect to conservative constituents will gradually approach the water-quality conditions predominating in the Caloosahatchee River. Although applicable in a rigorous sense only to conservative constituents, the equation also can be considered a fair first-order estimate for nonconservative constituents, especially along the primary flow paths through the system created by the tide gates (figs. 22 through 25). In this sense, the tide gates can be viewed as having formed parallel extensions of the Caloosahatchee River with a potential for establishing additional healthy and productive estuarine communities in much of the canal system.

A continuous supply of Caloosahatchee River water along the primary flow paths from the San Carlos system to the Bluejay system is likely to have some effect on both the average salinity and the range of salinity experienced in each system. In 1986, Douglas Morrison (City of Cape Coral, written commun., 1987) measured a salinity range of 10 to 29 ppt in the Bluejay system and 1 to 25 ppt in the San Carlos system. The difference is caused by inflow from the upstream freshwater canal system into the San Carlos system. The Bluejay system receives no inflow from this source. Installation of tide gates is expected to add sufficient saltwater to increase the average salinity and decrease the salinity range in the San Carlos system. The Bluejay system will receive additional saltwater and freshwater resulting in similar conditions.

Some degree of vertical density difference is often found in many parts of the study area. Water-quality data collected in the Bluejay system during June 1986 (table 1) show top-to-bottom differences in specific conductance from 1.5 to 12.0 mS/cm, an indication of increasing water density with depth. In February 1987, several sites in both the Bluejay and San Carlos systems (fig. 13) were found to have significant vertical density differences, as measured by specific conductance. Vertical density differences also were present at some time at most sites during the seasonal measurements shown in figures 11 and 12. Data collected in the study area by Douglas Morrison (City of Cape Coral, written commun., 1987) show that density stratification seems most persistent in the deeper canals and lakes where vertical mixing from water advection or surface winds is least effective.

Probably the most significant water-quality aspect of limited vertical mixing and associated density stratification is the tendency to isolate the deeper, more salty water from the atmosphere, which limits the replenishment of dissolved oxygen from the surface as it is consumed by oxygen-demanding bottom materials. An important question related to this study is the effects of tide gates on the formation and persistence of density stratification and low dissolved-oxygen conditions in the study area. This question cannot be answered with certainty, but available data do allow some reasonable inferences. Data collected primarily by Douglas Morrison (City of Cape Coral, written commun., 1987) during the summer of 1986 and some supporting data from this study show, not unexpectedly, that surface salinity in the Caloosahatchee River is often greater than bottom salinity in stratified lakes and canals within the study area. This oxygenated and more dense Caloosahatchee River water is expected to either displace or mix with the oxygen-depleted bottom waters encountered along the flow paths in the study area and thus increase bottom dissolved-oxygen concentrations. For less frequent conditions, when Caloosahatchee River surface salinity is less than that of canal system bottom water, the pulsing movement (caused by tide-gate operation), of oxygenated, less dense water over the oxygen-depleted layer may induce some oxygen entrainment and at least limit the extent or severity of dissolved-oxygen depletion.

Increased reaeration also can be expected to increase dissolved-oxygen concentrations in parts of the study area. Because reaeration generally increases with water velocity (Rathbun and Bennett, 1972), locations that have large velocity increases due to tide-gate operation will experience up to perhaps an order of magnitude increase in the capability to transfer oxygen from the atmosphere to the water. The same will occur at the tide gates; high velocities will induce accelerated reaeration.

The fate of stormwater runoff and associated constituents also will be affected by tide-gate operation. Stormwater and dissolved constituents that enter the primary flow paths through the canal system will be conveyed out of the canals and into the Caloosahatchee River in roughly the same timeframe as the advective period (5-10 days). Fine suspended material that requires a longer time period for sedimentation also will be flushed. Larger particles will be deposited on canal and lake bottoms, as occurs under present conditions. Stormwater that does not enter a primary flow path also will be dispersed relatively slowly, and much of the particulate load can be expected to settle to the bottom, as occurs at present.

Potential problems that may be caused by tide gates, particularly to that part of the Caloosahatchee River that receives water and constituents flushed from the canals, are predominately related to tide-gate startup. Tide gate B and the nongated interconnection at D (fig. 25) are at locations that link dead-end canals that have large bottom accumulations of fine materials. These accumulations probably will be resuspended with the startup of tide-gate operation if they are not physically removed prior to startup. These materials are likely to contain contaminants that are at present largely isolated (buried) from the aquatic environment and not subject to hydraulic resuspension into the water column and transport to other sites.

On the basis of results of the model and all the restricting assumptions, 28 percent of the material dissolved or suspended in the water column at the time the first tide gate goes into operation will be flushed into the Caloosahatchee River in the first 5 days. Over the next 45 days, another 26 percent will be flushed. Possible detrimental effects on Caloosahatchee River water quality are expected to be minimal, based on a comparison of the volume of river water tidally oscillating past the mouth of the canal systems, the receiving water, and the volume of original canal water discharged during the first 50 days of tide-gate operation.

On the basis of tidal prism concepts with a tidal range of 1.5 ft, the approximate volume of receiving river water that alternately floods and ebbs past the canal mouths is conservatively estimated at 1.1×10^{11} ft^3. The volume of original canal water entering this receiving volume during the first 50 days is about 1.2×10^{8} ft^3, or about 0.1 percent of the receiving volume. With nearly 100 tidal events during this 50-day period, mixing of canal and river water due to tide alone is likely to be nearly complete. If mixing is assumed to be only 10 percent effective, water-quality changes to the receiving water due to tide-gate startup will still be negligible. Measurable effects of canal discharge are likely to be concentrated in close proximity to the mouth of the Bluejay canal system. Because all water flushed from the study area will be replaced by river water, the effect of tide-gate startup on even localized areas of the Caloosahatchee River can be further minimized by assuring that canal and river water quality are as similar as possible.

In a similar vein, but recognizing the limitations of the one-dimensional assumption, the initial advective period of tide-gate operation (fig. 29) has the capability of discharging large quantities of oxygen-deficient water to the Caloosahatchee River. This can be avoided either by allowing only partial gate operation for a period of time, by starting operation during a period when the bottom water has a high dissolved-oxygen content, or by actively aerating the water column at points of discharge to the river.

After the startup period, canal and river water quality will be much the same, so there should be virtually no long-term detrimental effects. One possible exception is localized effects caused by stormwater runoff events. The potential benefits of improved water quality in miles of canal systems, however, are thought to more than offset this detriment.

In summary, the water-quality benefits that can reasonably be expected from tide-gate operation include (1) stabilization of salinity within the study area, (2) increased vertical mixing with reduction of density stratification and associated dissolved-oxygen depletion in canal bottom water, (3) increased local reaeration capability, and (4) increased discharge efficiency of stormwater runoff from the canals. Potential water-quality detriments are mostly related to the period of tide-gate startup and include (1) possible resuspension and transport of fine materials and associated toxics already accumulated on the bottom of some canals, (2) possible slug loading of oxygen-deficient water and other dissolved and suspended material to the Caloosahatchee River upon initiation of tide-gate operation, and (3) more efficient discharge of stormwater runoff to the Caloosahatchee River. On balance, the water-quality benefits accruing from tide-gate operation are expected to significantly outweigh the potential detriments.

SUMMARY AND CONCLUSIONS

This report documents an investigation (1) to determine the probable increase in circulation and flushing that can be induced by construction of tide gates in a part of the canal system of Cape Coral, Fla., and (2) to evaluate the probable water-quality effects on the canals and the adjacent Caloosahatchee River that could result from increased circulation and flushing.

The approach taken to accomplish these objectives included a combination of field data collection to define the physical and chemical characteristics of the study area and the application of numerical models to estimate circulation and flushing changes resulting from the construction of various combinations of tide gates and canal interconnections. Field data collected included fathometry, tidal stage, tidal discharge, and selected water chemistry throughout the study area. Circulation changes were projected using a one-dimensional, branched, hydrodynamic model capable of accurately simulating tidal and residual water motion in canal systems. Flushing changes were projected using hydrodynamic model results as input to a Lagrangian constituent-transport model. Estimates of water-quality changes caused by tide-gate construction were determined by concurrent evaluation of the model results and the observed water-quality characteristics of the study area.

The City of Cape Coral contains about 400 mi of dredged residential canals, of which 130 mi are tidally affected. This study focuses on two parts of the tidal canal system, the San Carlos and the Bluejay sections, that total about 29 mi in length. Each has only one tidal opening to the Caloosahatchee River. The two systems consist of canals varying mostly from 100 to 120 ft in width except near the Caloosahatchee River where 200-ft widths are common. Canal depths range from a foot or so at some dead ends to about 30 ft in the lowest lying areas near the river. Typical depths are from 5 to 10 ft. The canals are bridged at several sites to carry vehicular traffic and there are several very wide canal segments (called lakes). The bridges are areas of flow constrictions that are simulated closely by the models used. The lakes violate one-dimensional assumptions and may, therefore, be influenced by processes not included in the models used in this study.

Tidal stage measurements in both the San Carlos and the Bluejay canal sections show substantial differences. The spring tidal range in the Bluejay section is about 2 ft and only a little more than 1 ft in the San Carlos section. A phase lag also was measured between the two sections. The net result of these measurements is that water-level differences of 0.5 ft or more commonly exist between the two sections. The potential energy available in these stage differences can be converted to kinetic energy sufficient to increase water circulation and constituent flushing.

Tidal discharge data collected in both canal sections indicate relatively large rates of flow near the Caloosahatchee River and decreasing flows at sites progressively more inland. The data also indicate heavy localized runoff due to intense thunderstorms.

The general water-quality characteristics of the study area were determined by measurement of water temperature, specific conductance, dissolved oxygen, and pH at seven sites over a 20-month period. More detailed information was gathered during intensive sampling efforts in June 1986 and February 1987. Results of these measurements indicate canal water quality is influenced by Caloosahatchee River water, local stormwater runoff, and biochemical processes within the canals themselves. Water quality in the study area was found to be generally suitable for most purposes with some notable exceptions. In areas of density stratification, low dissolved-oxygen concentrations were commonly found in the bottom waters. These areas generally occurred in basins of deeper water flanked by shallows, such as in lakes and between bridges. Elevated nutrient concentrations also were associated with these areas.

Physical canal data were used to construct a numerical hydrodynamic model of the study area. Measured tidal stages at the boundaries were used to drive the model, and measured tidal stage and discharge were used to calibrate and verify the model's ability to simulate tidal action in the real canals. Excellent agreement between model and prototype was attained; the average nominal standard error between measured and computed stage at four sites and discharge at five sites was about 3 and 4 percent, respectively.

After verification, the hydrodynamic model was modified to allow simulation of from 1 to 11 tide gates and was applied about 150 times to determine optimum placements of from 1 to 4 tide gates for increased circulation and flushing in the canal system. The optimization criterion chosen was the volume-weighted average of tide-induced circulation in each segment of the model. This method takes into consideration that water in the lakes is of lesser quality than much of the rest of the canal system. The lakes are characterized by large volumes and sluggish water movement and circulation.

Results of model application indicate that construction of one tide gate having a width of 10 ft and a water depth of 5 ft will induce water circulation averaging about 168 ft^3/s through a part of the study area. Simulations indicate that use of two gates increases the total average circulation to 247 ft^3/s and expands the region of influence of increased circulation. The third and fourth connections increase circulation to 258 and 279 ft^3/s, respectively. The first gate is the most effective; each successive addition provides diminishing increases in circulation. At the maximum circulation rate, a volume of water equivalent to the total water volume within the study area will be circulated in about 9 days.

A one-dimensional, Lagrangian, constituent-transport model was applied to determine the effectiveness of tide gates to flush the San Carlos and the Bluejay canal systems. Model results indicate rapid replacement of canal water with river water during the first 7 to 10 days of tide-gate operation. This time period is needed to displace the original canal water along the primary flow paths through the canal system. Depending on the number of tide gates, the amount of canal water that remains in the system at the end of this advective period ranges from about 72 to 42 percent. With no tide gates, about 92 percent of the original canal water will remain.

After the advective period, the rate of replacement of canal water with river water slows markedly because canals adjacent to the primary flow path are flushed by dispersive rather than advective processes. Results from the model indicate that 50 days after the start of tide-gate operation, dispersion has reduced the amount of original canal water in the system to 47 percent with one gate and 21 percent with three gates and one interconnection. With no tide gates, about 73 percent of the original canal water is computed to remain after 50 days.

Results from the model given in this report are not applicable to determine changes in concentration of nonconservative constituents, such as dissolved oxygen, nutrients, suspended sediment, or phytoplankton. To develop a predictive model capable of reliably estimating changes in all these variables would require considerably more information on the many rate processes involved. Such processes (among others) include rates of input, sediment oxygen demand, reaeration, photosynthesis, respiration, nutrient exchange, nitrification, denitrification, and grazing, as well as sediment resuspension, transport, settling, deposition, and compaction. Even without such information, however, reasonable inferences regarding some effects that tide-gate construction might have on nonconservative water-quality constituents are:

- Reducing density stratification and associated dissolved-oxygen depletion in canal bottom waters,
- Increasing localized reaeration capability, and
- Providing for more efficient discharge of stormwater runoff from canals.

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