Simulation of the Water-Table Altitude in the Biscayne Aquifer, Southern Dade County, Florida, Water Years 1945-89

United States Geological Survey Water-Supply Paper 2458

Prepared in cooperation with the Metro-Dade Department of Environmental Resources Management

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By Michael L. Merritt

Prepared in cooperation with the
Metro-Dade Department of Environmental Resources Management

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[Plates are in pocket]

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CONVERSION FACTORS

Multiply	By	To obtain
inch (in.)	25.4	millimeter
inch per day (in/d)	25.4	millimeter per day
inch per year (in/yr)	25.4	millimeter per year
foot (ft)	0.3048	meter
foot per day (ft/d)	0.3048	meter per day
foot per second (ft/s)	0.3048	meter per second
foot per mile (ft/mi)	0.1894	meter per kilometer
feet per foot (ft/ft)	0.3048	meters per meter
foot squared per day (ft^2/d)	0.09290	meter squared per day
cubic foot (ft^3)	0.02832	cubic meter
cubic foot per second (ft^3/s)	0.028317	cubic meter per second
cubic foot per day (ft^3/d)	0.028317	cubic meter per day
acre-foot per year (acre-ft/yr)	0.001233	cubic hectometer per year
mile (mi)	1.609	kilometer
square mile (mi^2)	2.590	square kilometer
pound (lb)	4.636	kilogram
pound per day (lb/d)	4.636	kilogram per day
pound per square inch (lb/in^2)	6.895	kilopascal
pound per cubic foot (lb/ft^3)	16.02	kilogram per cubic meter
gallon per day (gal/d)	0.003785	cubic meter per day

Temperature in degrees Celsius (°C) can be converted to degrees Fahrenheit (°F) as follows:

$$°F = 1.8\ (°C) + 32$$

Sea level: In this report, “sea level” refers to the National Geodetic 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.

Abbreviations and Acronyms

mg/L milligrams per liter
SASS surficial aquifer system study
SWIP Subsurface Waste Injection Program
USGS U.S. Geological Survey

Simulation of the Water-Table Altitude in the Biscayne Aquifer, Southern Dade County, Florida, Water Years 1945-89

By Michael L. Merritt

Abstract

A digital model of the flow system in the highly permeable surficial Biscayne aquifer of southern Dade County, Florida, was constructed for the purposes of better understanding processes that influence the flow system and of supporting the construction of a subregional model of the transport of brackish water from a flowing artesian well. Problems that needed resolution in this endeavor included the development of methods to represent the influence of flowing surface water in seasonally inundated wetlands and the influence of a network of controlled canals developed in stages during the simulation time period (water years 1945-89). An additional problem was the general lack of natural aquifer boundaries near the boundaries of the study area.

The model construction was based on a conceptual description of the Biscayne aquifer developed from the results of previous U.S. Geological Survey investigations. Modifications were made to an existing three-dimensional finite-difference simulator of ground-water flow to enable an upper layer of the grid to represent seasonally occurring overland sheetflow in a series of transient simulations of water levels from 1945 to 1989. A rewetting procedure was developed for the simulator that permitted resaturation of cells in this layer when the wet season recurred. An “equivalent hydraulic conductivity” coefficient was assigned to the overland flow layer that was analogous, subject to various approximations, to the use of the Manning equation. The surficial semiconfining peat and marl layers, levees, canals, and control structures were also represented as part of the model grid with the appropriate choices of hydraulic coefficient values.

For most of the Biscayne aquifer grid cells, the value assigned to hydraulic conductivity for model calibration was 30,000 feet per day and the value assigned to porosity was 20 percent. Boundary conditions were specified near data sites having long-term records of surface-water stages or water-table altitudes, and modifications to the simulator permitted the specification of time-varying pressures at boundary grid cells. Rainfall data from a station in Homestead generally were used as an areally uniform rainfall specification throughout the modeled region. Maximum evapotranspiration rates ranged seasonally from a minimum of 0.08 inch per day in January to a maximum of 0.21 inch per day between June and October. Shallow-root and deep-root zone depths for the evapotranspiration calculation were 3 and 20 feet in the coastal ridge and were 0.10 and 5 feet in the glades regions where peat and marl covers occurred.

Results of sensitivity analyses indicated that the simulations of stages and water levels were relatively unresponsive to 50 percent changes in aquifer hydraulic conductivity, porosity, and the equivalent hydraulic conductivity of overland flow. However, 20 percent changes in rainfall and maximum evapotranspiration rates produced significantly different water levels, as did interchange of coastal ridge and glades deep-root zone (extinction) depths.

Water levels were simulated very well at most measurement sites. Sensitivity analyses illustrated the significant influence of the uncontrolled agricultural drainage canals on pre-1968 regional water levels and the further influence of Black Creek Canal in draining a region of high water after 1961. Other analyses indicated that the flood-control system of 1968-82 lowered peak water levels in the affected region by as much as 1.5 feet in the wet summers of 1968, 1969, and 1981, and that Levee 67 Extended channeled flows from the S-12 spillway structures and raised overland flow stages in Shark River Slough. Hypothetical scenarios of well-field pumping in the vicinity of Levee 31N indicated that the pumping induced a significant amount of recharge from the adjacent borrow canal, the degree of which depended on the distance between the canal and the well field. The computed ratio of evapotranspiration to rainfall recharge ranged from 88 to 94 percent during water years 1945-82. The ratio increased to about 97.9 percent during water years 1983-89, possibly because of changing water-management practices and deficient rainfall.

INTRODUCTION

Abundant supply and massive demand have characterized the system of municipal water deliveries in Dade County, Fla. (fig. 1). Dade and parts of neighboring counties are underlain by a surficial aquifer system that includes the Biscayne aquifer, one of the most productive in the Nation (Fish and Stewart, 1991). No other source in Dade County is used for public-water supply, and consequently, the Biscayne aquifer was designated a sole-source aquifer by the U.S. Environmental Protection Agency in 1979.

The carbonate Biscayne aquifer extends to land surface in many parts of the area, or is overlain by thin layers of various materials that permit rapid infiltration from land surface. This is advantageous in permitting rapid replenishment of the aquifer, but is also disadvantageous because surface spills of liquid contaminants can rapidly enter the aquifer. In the urbanized eastern part of Dade County, such spills have occurred with serious consequences for public-water supply when contaminants in the aquifer have traveled toward well fields. The former Medley Well Field (fig. 2), in an industrial area west of Hialeah, was abandoned in 1983 for this reason (Technical Advisory Committee for the Proposed West Well Field, written commun., 1988). Pumping from wells at the Hialeah, John E. Preston, and Miami Springs Well Fields, which have provided a major part of water used in the greater Miami metropolitan area, has been curtailed at times because of the presence of contaminants from industrial sources upgradient, and air stripping is now a standard part of the treatment process to remove organic pollutants.

The southern service area of Dade County has undergone an especially rapid increase in population and water demand. Water for the Miami-Dade Water and Sewer Authority Department, southern district, is supplied by the Alexander Orr, Snapper Creek, and Southwest Well Fields (fig. 2). All are in urban areas and are subject to industrial contamination. In 1982, contaminants detected in the soil near the Alexander Orr Well Field and water-treatment plant had to be removed.

Because of the increasing water demand and potential for contamination, Dade County has proposed development of an additional well field to supply the southern service area. The proposed West Well Field is to be located inland, west of urban development and potential sources of industrial contamination, and will be connected by a pipeline with the Alexander Orr Water Treatment Plant. Various sites in bordering wetlands have been considered (Technical Advisory Committee for the Proposed West Well Field, written commun., 1988), all of which are in the wetlands east and west of Levee 31N and south of Tamiami Trail (fig. 2).

Even in these wetlands, however, a possible source of water-supply contamination exists in the form of a plume of saline water originating from a formerly flowing artesian well (fig. 2). The flowing well was located in Chekika State Recreation Area, a 1-mi^2 section that includes Grossman Hammock, an area of relatively high elevation covered with natural hardwood vegetation. The “Grossman well” (fig. 2) was drilled in Grossman Hammock in late 1944 as part of an oil exploration effort. The 1,200-ft (foot) well penetrated permeable strata of the artesian Upper Floridan aquifer, which is separated from the Biscayne aquifer by a thick confining layer, and brackish water flowed from the well and entered a recreational lake. The lake overflowed into a deep borrow pit, which allowed the

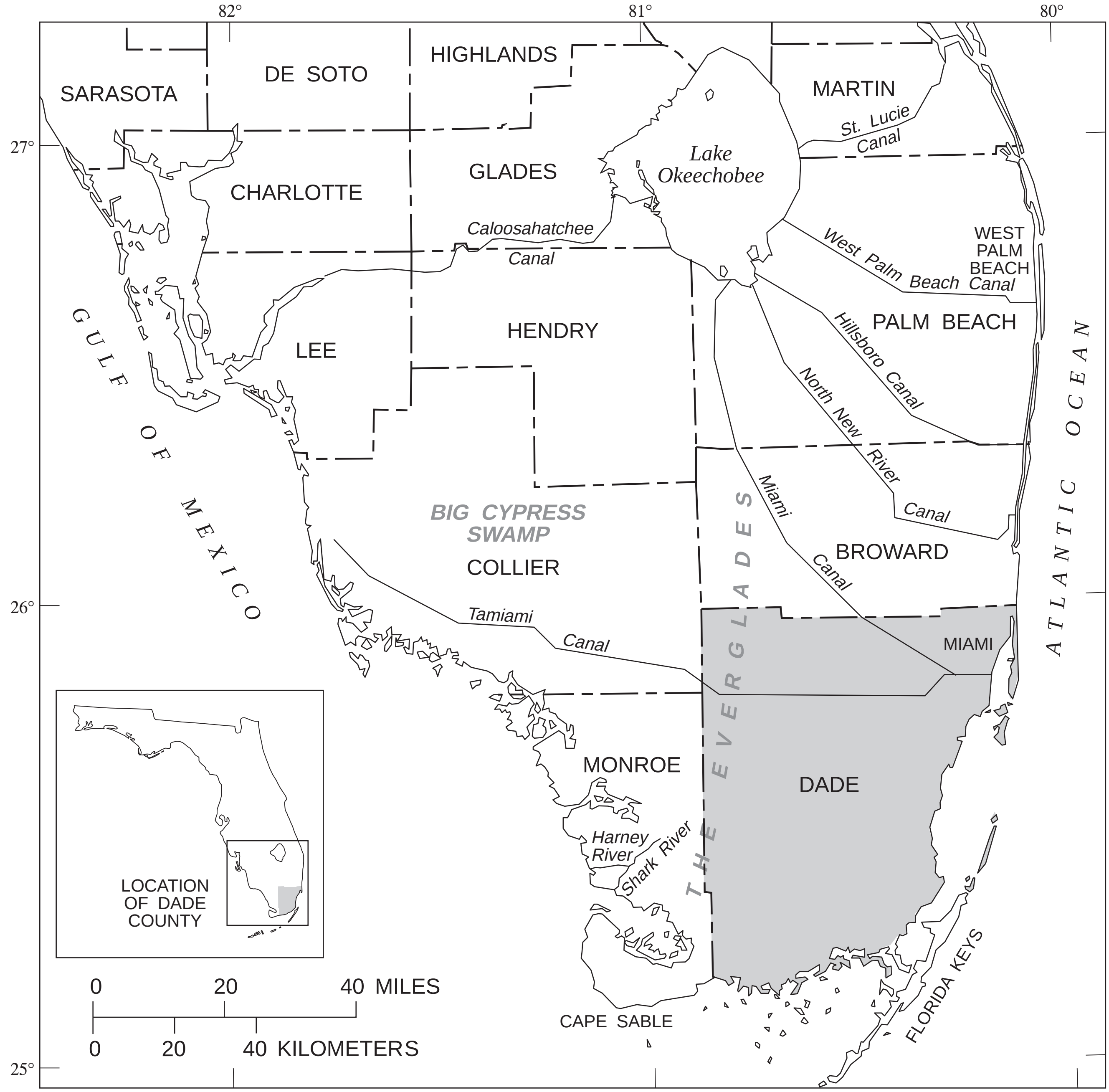


Figure 1. Locations of counties, major physiographic featues, and major regional drainage canals in southern Florida.

brackish water to infiltrate the Biscayne aquifer. A reconnaissance of area water quality (Waller, 1982) delineated a plume of brackish water extending about 8 mi (miles) southeastward from the well. The proximity of the plume to the proposed site for the West Well Field raised the possibility that water from the well field might need additional treatment if contaminated by brackish water from the flowing well. The Metro-Dade Department of Environmental Resources Management began a program of water-quality monitoring in the Biscayne aquifer near the plume (Labowski, 1988) in February 1982 and the Grossman well was plugged in March 1985.

To obtain better information concerning the future movement and degree of dissipation of the plume and effects related to the hydraulic influences of a nearby well field, the U.S. Geological Survey (USGS) entered into a cooperative agreement with the

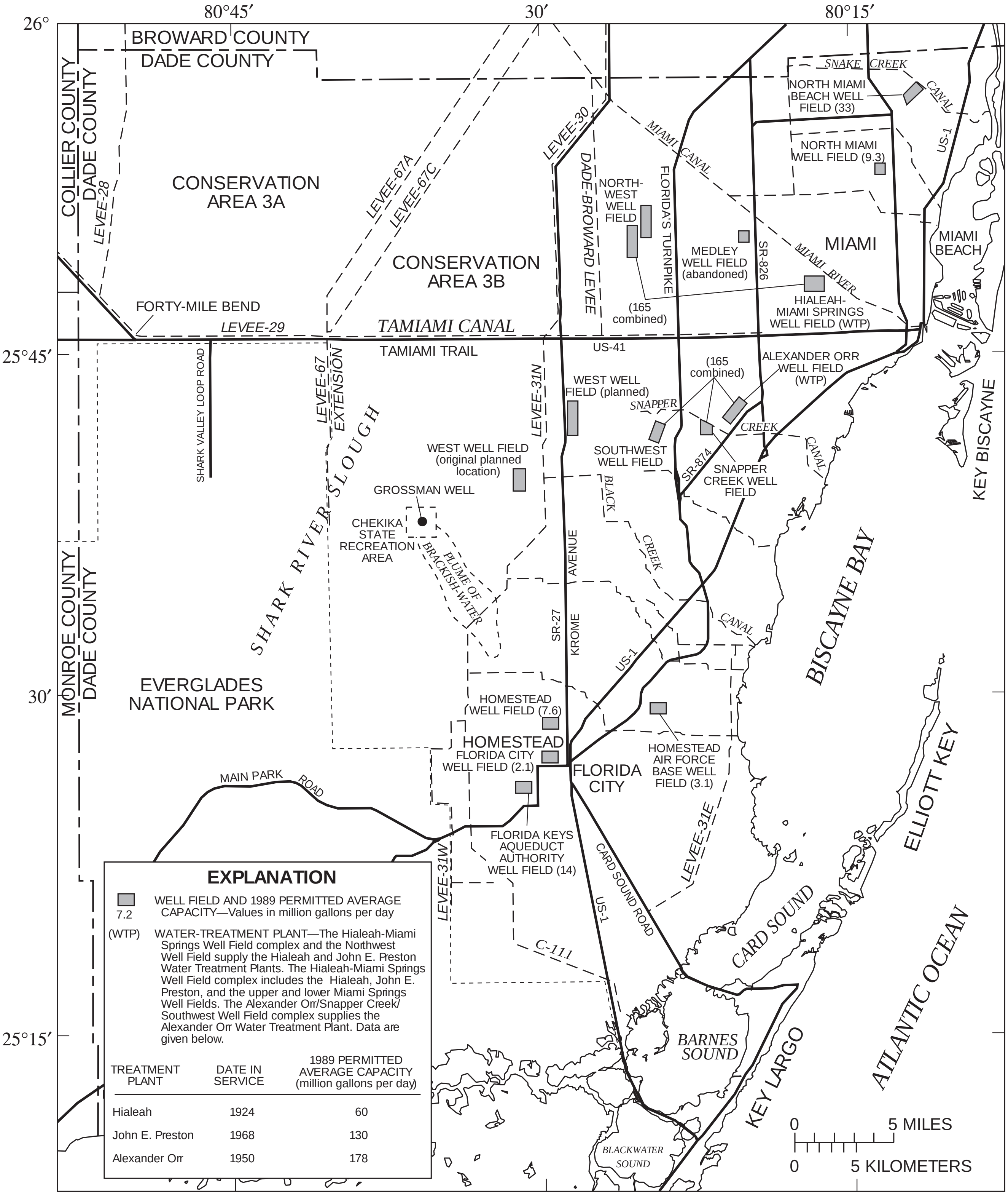


Figure 2. Locations of major well fields and water-treatment plants in Dade County.

Metro-Dade Department of Environmental Resources Management to conduct a study of the plume development and behavior. It was recognized that a detailed analysis could be made by using digital modeling techniques and that simulating the development of the plume required a realistic representation of the natural flow regime in the area. However, simulation of regional-scale surficial aquifer flow in southern Florida presents special challenges and difficulties, particularly when transient conditions are to be represented. Such difficulties have reduced the number of large-scale surficial aquifer flow-model applications attempted either by public agencies or private firms, despite the importance of the aquifer and the existence of numerous local contamination hazards.

One challenge to the precise modeling of the flow system is the lack of natural hydraulic boundaries. Commonly, canals are used as time-invariant boundaries for models in southern Florida. In some parts of the area, however, canals are in direct connection with the Biscayne aquifer, and canal stages are largely controlled by surrounding ground-water levels. Canal stages are also marked by periodic spatial discontinuities at control structures that are opened or closed according to complex schedules.

The system of canals and control structures now in operation substantially alters the natural ground-water flow system. The canal system was developed in stages during the period in which the Grossman well discharged saline water to the aquifer. Thus, a simulation of the ground-water flow system for the purpose of modeling plume development requires a consideration of changes in the hydraulic regime caused by the gradual addition of water-management control features.

Another difficulty facing investigators attempting to model parts of the Biscayne aquifer flow system is the widespread, seasonal flooding of wetlands, such as the marshes of the Everglades and the coastal glades. At the time this study began (1986), no known USGS-supported flow models were available for the transient simulation of combined surface-water and ground-water flow systems that include extensive areas of overland sheetflow.

Other aspects of surficial aquifer hydrology in Dade County that present challenges for modeling are the high permeability and resulting shallow hydraulic gradients, and the closeness of the saturated zone to land surface, which causes a strong and rapid response of water levels to atmospheric recharge and evapotranspiration. Because of the scope and complexity of the technical issues involved, the first phase of the plume study was directed toward the development of a regional flow simulation to resolve the cited simulation difficulties.

Purpose and Scope

This report documents the development and testing of a computer simulation of the water-table altitude in the Biscayne aquifer in the region of Dade County south of the Tamiami Trail (fig. 2). The documentation includes a description of the resolution of simulation problems that are a consequence of natural hydrologic features of the flow system and that are posed by introduced water-management controls.

To provide the basis for model construction and the use of selected methods, the geohydrology and surface-water hydrology of the area are described, with special emphasis on the significance of surface-water and ground-water interaction and features of the water-management system. The report describes separate simulations designed for each of five successive time periods between water years 1945 and 1989 representing distinct evolutionary stages in the development of water-management controls. The simulations are evaluated using sensitivity analysis techniques to assess the significance of parameters representing natural hydrologic processes and introduced water-management features. Results of these analyses are used as a basis for limited interpretations concerning the natural hydrologic controls on the ground-water flow system and how construction for water-management purposes has modified the system. The study is based almost entirely on data collected as part of the basic hydrologic data program of the USGS and data collected by other governmental agencies. Detailed descriptions of mathematical modifications to the selected computer code and use of the flow model for analysis of the movement of a high-chloride plume in southern Dade County (the study area) have been documented separately.

Previous Studies

The Biscayne aquifer of southeastern Florida has received considerable attention in many scientific studies. The first major comprehensive study by Parker and others (1955) documented predevelopment conditions and features of the early water-management system. A general description of the hydrology of the Biscayne aquifer and potential contamination hazards was provided by Klein and Hull (1978) in support of the petition for sole-source designation by the U.S. Environmental Protection Agency.

In 1979, the USGS began a comprehensive study to delineate the geologic, chemical, and hydraulic characteristics of the surficial aquifer system, which includes the Biscayne aquifer. A program of dual-tube reverse-air drilling and data collection in Dade County was conducted in 1983-84. A geologic interpretation, including detailed descriptions of lithology at each well and formation correlations along east-west and north-south sections, was prepared by Causaras (1987), and water-quality aspects were reported by Sonntag (1987). A comprehensive report on results of hydraulic testing in the surficial aquifer system study (SASS) was prepared by Fish and Stewart (1991), who also cited data obtained from previous investigations in Dade County and from public-supply well testing, and provided a comprehensive delineation of the surficial aquifer system in Dade County. The simulation described herein is based to a substantial extent on the data provided by Fish and Stewart (1991).

Additional data describing geohydrologic conditions in the region containing the chloride plume were provided by a study of the hydrology of the East Everglades, an area of inland wetlands west of the urbanized coastal ridge of Dade County and bordering Everglades National Park (Schneider and Waller, 1980; Waller, 1983). Leach and others (1972) focused on hydrologic effects of parts of the water-management system constructed before 1970. Klein and others (1975) presented a comprehensive description of the ecology and hydrology of southern Florida in support of a Federal and State effort to develop a land-use policy for the region.

The representational difficulties related to surface-water and ground-water flow interaction have largely forestalled the USGS from participating in studies that include regional-scale simulation modeling of flows in the surficial aquifer system. The most recently documented USGS study that involved a simulation of Biscayne aquifer flows was conducted by Appel (1973), who used an electrical analog model. Cordes and Gardner (1976) documented an application of the analog model to two water-management scenarios, and Klein (1976) reported on the use of the analog model to simulate well-field pumping scenarios.

More recently, the South Florida Water Management District, working under the sponsorship of the U.S. Army Corps of Engineers, coupled a ground-water flow solution to a surface-water routing model for the purpose of simulating the surface flow systems (including canals and wetlands) of the upper and lower southeast coasts of southern Florida and their effect on the water table (MacVicar and others, 1984). The solution is driven by inputs of rainfall and canal flows measured at control structures. After stage changes in surface-water bodies are computed, estimates are made of sources and sinks affecting storage in the ground-water system, including seepage to or from canals and wetlands, rainfall recharge, and evapotranspiration. Changes in water-table altitude are then computed by an explicit solution to a two-dimensional ground-water flow equation similar to that solved by Trescott and others (1976). In 1991, plans were developed to upgrade the capabilities of the model for possible application to a region as large as the southern part of the Florida Peninsula.

Private firms have applied ground-water flow modeling techniques to address concerns about the management of the surficial aquifer system in Dade County. Camp, Dresser, and McKee (1981) simulated traveltime contours around well fields in Dade, Broward, and Palm Beach Counties (fig. 1), using a “random walk” solute-transport model (Prickett and others, 1981). The approach was to simulate an annual average set of hydraulic conditions, both regionally and in a subregional model with higher grid resolution and containing pumping wells, and then to use that information to simulate the hydraulic regime and drawdowns that would result from a worst-case drought period of 210 days. The resulting head gradients were inverted, and particles representing a tracer were injected at pumping well locations. The positions of various particle concentrations at subsequent times were observed in order to obtain traveltime contours. The regions modeled did not include wetlands. Bordering wetlands provided heads to be used as specified time-invariant boundary conditions. Canals were represented with a river package similar to that of Trescott and others (1976) that assumed canal stages to be time invariant.

Ground-water flow models have been used to simulate drawdown effects from pumping at various individual well fields in southeastern Florida.

The cone of influence around the Northwest Well Field (fig. 2), defined as the 0.25-ft drawdown contour, was simulated by Camp, Dresser, and McKee (1984) using a generic flow model code (Prickett and Lonnquist, 1971), again representing a 210-day drought period in a subregional model with generalized boundary conditions at some distance from the well-field site. The model was calibrated under steady-state and short-term transient conditions. Simulated drawdowns were reported to be especially sensitive to canal stages and the canal recharge rate. The latter is the rate of inflow to the aquifer from the canal, which is considered to have a time-invariant stage specified by the user, and the inflow rate is determined by a user-specified leakage coefficient.

More recently (1987), Camp, Dresser, and McKee has provided an unpublished proprietary code (DYNEFLO) to the Metro-Dade Department of Environmental Resources Management for use in simulating potential drawdowns from the proposed West Well Field. The three-dimensional finite-element simulator has been calibrated to provide a two-dimensional representation of water levels in all of Dade County. The basic representational methods of the finite-difference code used earlier by Camp, Dresser, and McKee (1981) are also present in the application of the newer code.

Because of concerns that effects of overland sheetflow and canal leakage on ground-water levels and their relation to potential well-field drawdowns might not be adequately represented by the DYNEFLO model application, additional studies have been supported by the Technical Advisory Committee for the Proposed West Well Field (written commun., 1988) and by other agencies of Dade County. In one such study, Chin (1990) assessed canal gains from aquifer storage and losses to the aquifer with a section of the L-31N canal as a specific study area.

Acknowledgments

This study owes its initiation partly to the interest of James L. Labowski, formerly with the Metro-Dade Department of Environmental Resources Management, who believed that research into the extent and future movement of the high-chloride plume originating from the flowing Grossman well would add significantly to understanding the hydrogeology of the Biscayne aquifer, thereby providing important assistance to water managers.

Appreciation is extended to David Sikkema of the Everglades Research Center, Everglades National Park, for supplying stage and water-level data for calibration and for providing orientation on water-management issues relating to the park. Useful information and data were obtained from Ron Mireau, John Adams, Dick Slyfield, and various other personnel of the South Florida Water Management District, Jim Vearil of the U.S. Army Corps of Engineers, the Institute of Food and Agricultural Sciences near Homestead, the National Hurricane Center in Coral Gables, Fla., and the National Oceanic and Atmospheric Administration in Asheville, N.C.

The author received considerable guidance in developing an understanding of the ground-water and surface-water hydrology of Dade County and the workings of the water-management system from Bradley G. Waller, formerly of the USGS. Waller’s expertise and continuing willingness to provide answers to various inquiries greatly facilitated the author’s progress. Appreciation is also extended to Charles A. Appel (USGS, Reston, Va.) and Mary C. Hill (USGS, Denver, Colo.), who provided thorough reviews of this extensive manuscript.

HYDROLOGIC CONDITIONS IN SOUTHERN DADE COUNTY

Previous descriptions of the natural hydrology of southern Dade County and hydrologic changes caused by anthropogenic activities are limited in scope or in geographical extent. For this reason, the subsequent sections are included to provide a discussion of hydrologic topics of particular significance to the construction of a regional model of ground-water flow and its relation to surface water.

Surficial Aquifer System

Properties of the Biscayne aquifer and overlying and underlying deposits comprising the surficial aquifer system are discussed on the following pages. The properties discussed are ones that require definition for the construction of a model of ground-water flow.

Stratigraphy

The hydrostratigraphic section in southern Dade County consists of a surficial aquifer system underlain by a confining unit that separates it from the Floridan aquifer system. In the study area, the surficial aquifer system correlates with the Miami Limestone, Anastasia Formation, Fort Thompson Formation, and the Key Largo Limestone, all of Pleistocene age, and the underlying Tamiami Formation of Pliocene age. (The formation name, Miami Oolite, was changed to Miami Limestone by Hoffmeister and others (1967) in recognition that rocks of this formation had a bryozoan facies, rather than an oolitic facies, in many parts of the mainland of southern Florida.) The Pleistocene rocks comprise the Biscayne aquifer in most of southern Dade County. The top of the Hawthorn Formation of Miocene age that underlies the Tamiami Formation marks the base of the surficial aquifer system, and generally lies about 200 ft below land surface, though the top of the Hawthorn Formation is as shallow as 150 ft below land surface at some locations in the study area (Causaras, 1987).

Figure 3 shows southern Dade County divided into three regions and seven subregions according to the nature of the rocks or soils found at land surface (U.S. Department of Agriculture, 1947). The soil types in southern Dade County closely correlate with local features of the natural surface-water and ground-water system.

Region 1 is a triangular-shaped sector comprising the northwestern part of the study area. Region 1B is the central part of the Everglades wetlands in southern Dade County, usually referred to as Shark River Slough. The generalized geologic section for region 1 (fig. 4) shows that surface deposits are a layer of organic sediment (peat). The peat was deposited over a long period of time during seasonal periods when overland sheetflow occurred as a result of intense summer rainfall. The underlying layer of Miami Limestone generally is about 10 ft thick and of low permeability in this region. The Fort Thompson Formation underlying the Miami Limestone thins to the northeast as it loses the solution porosity that makes it highly permeable. The dashed line bisecting region 1 in figure 3 marks the approximate western limit of the Biscayne aquifer (Fish and Stewart, 1991).

Rocks of the Fort Thompson Formation are underlain by nearly 200 ft of the Tamiami Formation which contains a 40- to 80-ft section of permeable limestone (the Everglades aquifer or gray limestone aquifer) separated from the overlying Fort Thompson Formation and from the underlying Hawthorn Formation by clastic materials of low permeability. The upper part of the Hawthorn Formation typically is clayey and of very low permeability in this region.

Region 3 soils occur mostly in the southern part of the study area and extend up the east coast in a strip that narrows toward the north. Region 3 is similar to region 1 in that overland sheetflow occurs during part of the year, but is confined from underlying rocks by low-permeability deposits that are best characterized as calcitic muds. These deposits, to which the name Lake Flirt Marl has been applied, probably represent the cumulative deposition of a calcite-bearing periphyton (Gleason, 1984). The underlying Miami Limestone is about 10 to 15 ft thick in the central and eastern parts of the continuous southern part of region 3, but wedges out in the west near the Dade-Monroe County line (fig. 2). The Fort Thompson Formation underlies the Miami Limestone, as in region 1, thickening from about 10 ft in the west to nearly 60 ft in the east. The Tamiami Formation underlying the Fort Thompson Formation is 100 to 200 ft thick and consists entirely of low-permeability clastics. It is underlain by the Hawthorn Formation, the upper part of which consists of silty, clayey sands of low permeability.

Region 2 includes the urban and agricultural areas of southern Dade County that replaced the primeval pine forests in the 19th century and early part of the 20th century. Rocks of the Miami Limestone extend to land surface. A thin overlying layer of rocky soil in the eastern part, region 2B, does not appreciably retard downward percolation of rainfall. In region 2B, the southern extent of the Atlantic Coastal Ridge, land elevations range from 12 to 15 ft above sea level. Region 2B generally is not seasonally inundated. Adjacent southern and southwestern parts (region 2A) are slightly inundated during extended wet periods. The noninundated section of the coastal ridge extends westward into Everglades National Park, an area referred to as Long Pine Key.

In region 2, the Miami Limestone is 10 to 30 ft thick. The underlying Fort Thompson Formation increases in thickness from 20 ft in the west, near Region 1B, to 85 ft in the northeastern part of the study area. Farther to the northeast, the Fort Thompson is interbedded with the sandy Anastasia Formation and the coralline Key Largo Limestone. These two latter formations extend for only a short distance into

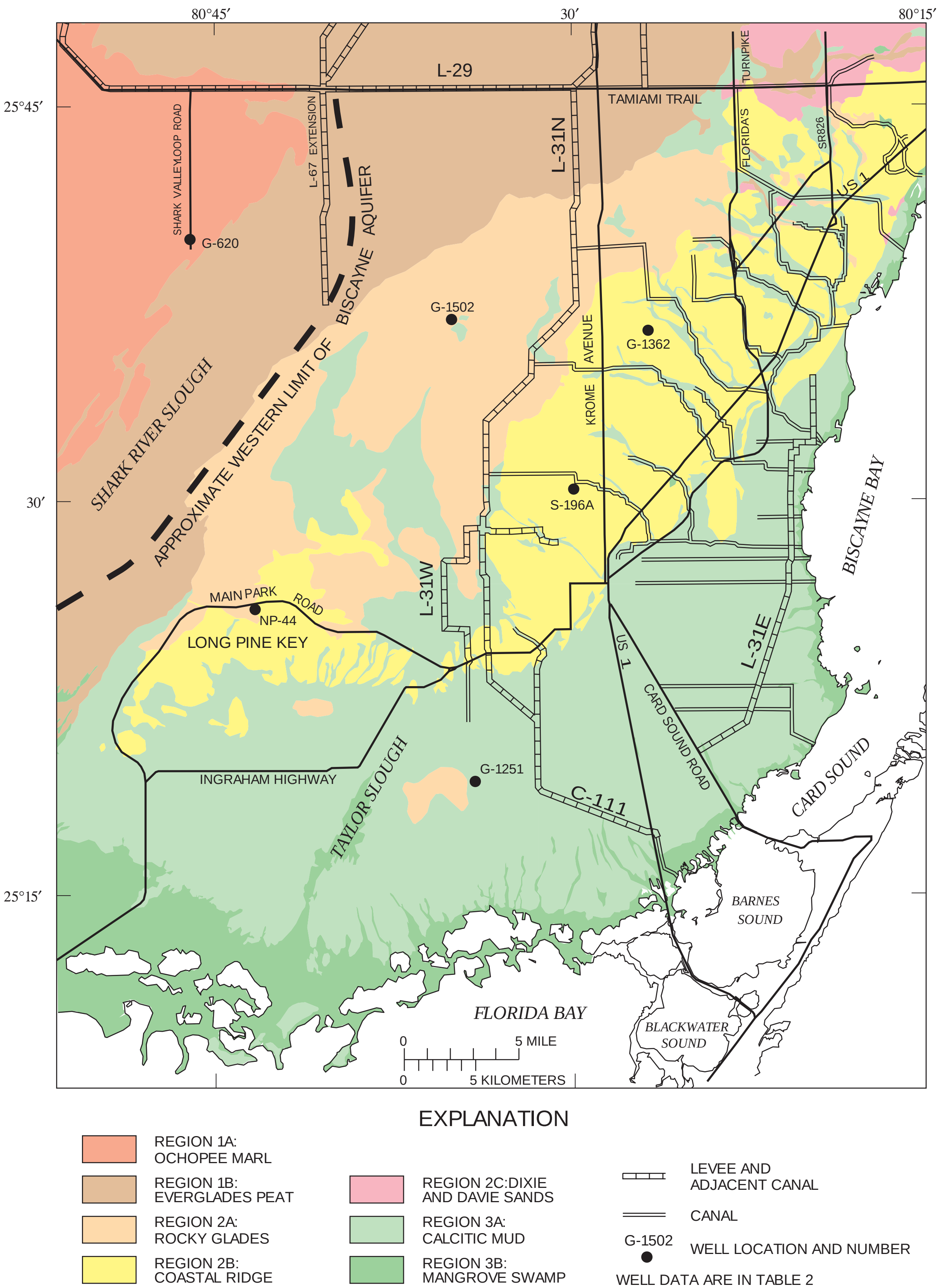


Figure 3. Surface soils of southern Dade County.

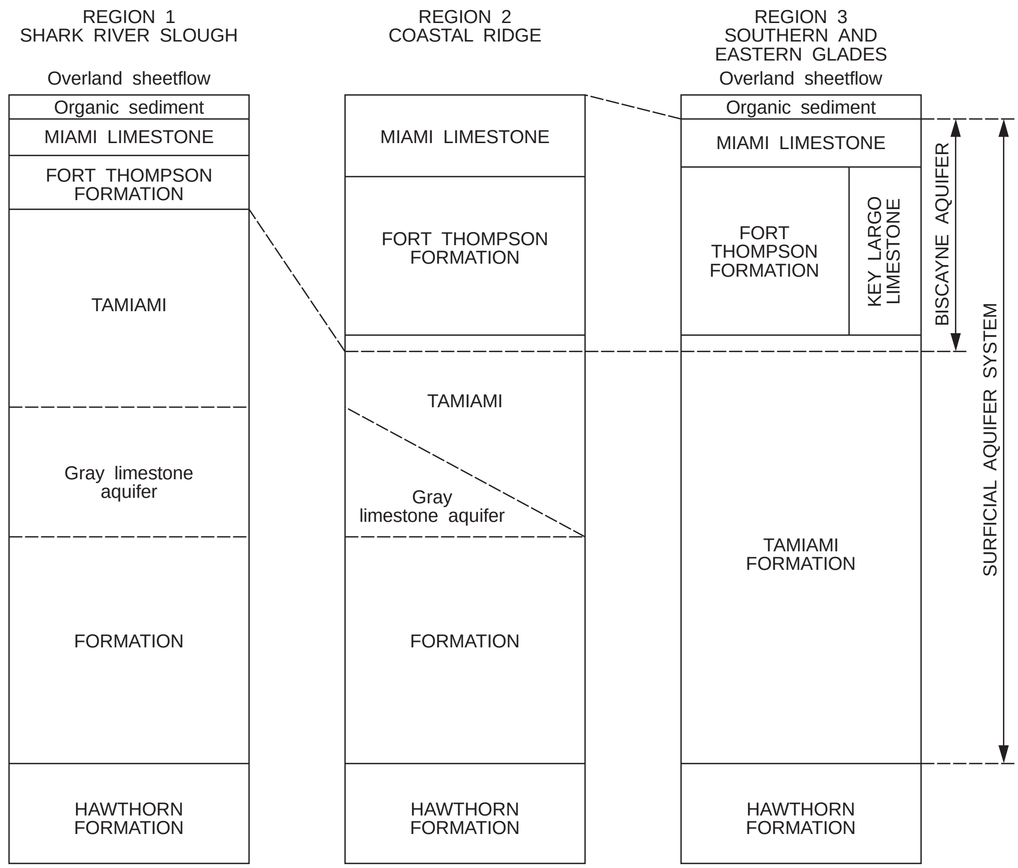


Figure 4. Generalized stratigraphy of formations comprising the surficial aquifer system in parts of Dade County.

the modeled area. Rocks of the Key Largo Limestone apparently replace the Fort Thompson Formation in extreme southeastern Dade County (in region 3B).

In the Tamiami Formation, which underlies the Fort Thompson everywhere in region 2, a section of moderate permeability of the gray limestone aquifer (Fish and Stewart, 1991) extends from region 1 into the northwestern part of region 2. Toward the southeast, the overlying confining layer of clastics (shelly sands with interbeds of green silty or calcareous clay) is reported to become leaky (Labowski and others, U.S. Geological Survey, written commun., 1988), and to contain less clay beds, and the gray limestone aquifer thins and has less solution porosity. Farther to the southeast, the aquifer material grades into clastic materials of low permeability. Along the coast of Biscayne Bay, the upper part of the Tamiami Formation is sufficiently permeable to have been included in the Biscayne aquifer by Fish and Stewart (1991). The Tamiami Formation is underlain everywhere in region 2 by low-permeability materials of the Hawthorn Formation.

The depth below sea level corresponding to the base of the Biscayne aquifer south of the Tamiami Trail is shown in figure 5. Generally, the depth is equivalent to the thickness of the section comprised of a few inches to a few feet of surface deposits and the underlying Miami Limestone and Fort Thompson Formation. However, the highly permeable upper deposits of the Tamiami Formation near the east coast are also included in the mapped thickness.

Permeability

Construction of a model of aquifer flows requires representation of the ability of the aquifer to transmit water under the influence of the prevailing hydraulic gradient. The general transmission property of the aquifer material, independent of fluid properties, is referred to as permeability. Permeability is considered to vary spatially within the aquifer. When a measure of the permeability is combined algebraically with the fluid properties of density and dynamic viscosity and expressed in terms of a unit thickness of

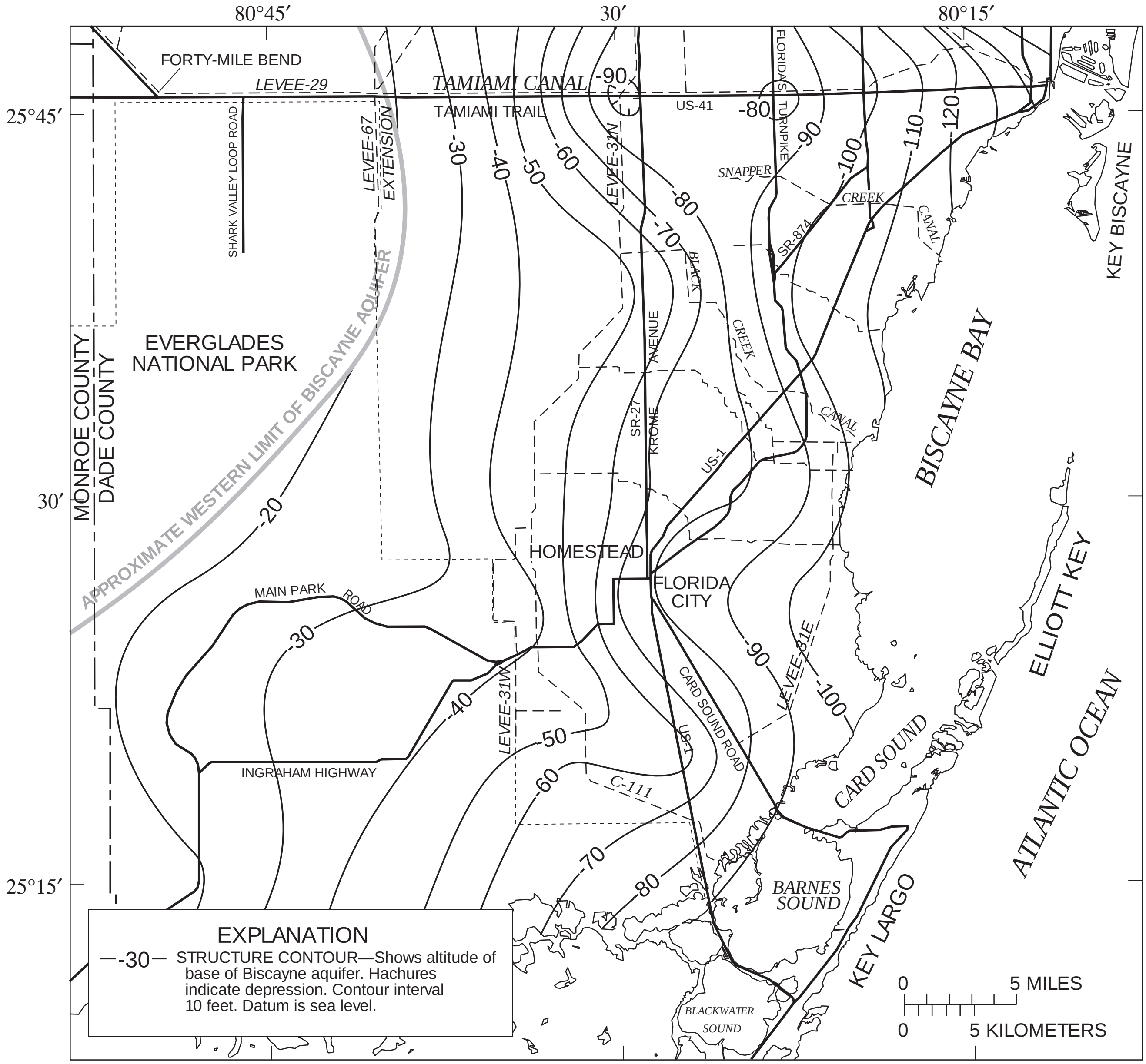


Figure 5. Configuration of the base of the Biscayne aquifer in southern Dade County.

aquifer, the resulting parameter is referred to as the hydraulic conductivity, having English units of feet per day, and is a measure of the ability of the aquifer to transmit fluid of specified properties under a specified hydraulic gradient. Hydraulic conductivity is the parameter used to compute rate of ground-water flow in three-dimensional or cross-sectional models and in models where the wetted thickness varies spatially or temporally. When hydraulic conductivity is integrated over the vertical thickness of the aquifer, the resulting parameter is transmissivity, in units of feet squared per day.

Field studies of aquifer properties generally measure the transmissivity of the aquifer. The USGS study of the surficial aquifer system in Dade County (Fish and Stewart, 1991) and previous studies provided estimates of transmissivity based on the results of single-well aquifer tests. Hydraulic conductivity estimates are often derived from transmissivity measurements based on the assumption that the hydraulic conductivity is vertically uniform throughout the vertical thickness of the aquifer. Such estimates are most precisely regarded as estimates of the average hydraulic conductivity. Independent estimates of the hydraulic conductivity of vertical intervals of the aquifer are generally only possible when there is a basis for independent estimates of the transmissivities of those intervals.

The peats of Shark River Slough and the calcitic muds (Lake Flirt Marl) of the southern and eastern wetlands are known to be of low permeability, though quantitative measurements are lacking. Where thick sections of the Miami Limestone and Fort Thompson Formation are present, they are characterized by a dense network of large solution features (Fish and Stewart, 1991) in which individual cavities are as large as 2 in. (inches) in diameter. Fish and Stewart (1991) report that Miami Limestone cavities are partly filled with sand and silt, reducing the permeability relative to that of the underlying Fort Thompson Formation. J.E. Fish (U.S. Geological Survey, oral commun., 1985) observed that high values of specific capacity obtained during tests of wells completed in the Miami Limestone as part of the surficial aquifer system study (SASS) were suspect because the wells were near canals, and the formation might have had local solution features of recent origin uncharacteristic of the aquifer on an areal basis. Observing that the formation could be successfully dewatered for construction projects at locations in the urbanized northeastern and east-central parts of Dade County, Fish suggested 5,000 ft/d (feet per day) as a reasonable estimate for the average hydraulic conductivity of the Miami Limestone. This estimate, however, lacks validation by a properly designed program of aquifer testing.

In contrast to this evidence are data from three sets of two or three wells drilled to different depths (10-15 ft deep) in the part of the Biscayne aquifer affected by the chloride plume from the Grossman well in Chekika State Recreation Area (fig. 2) that flowed under artesian pressure until it was plugged in 1985. Specific conductance and chloride concentrations do not change appreciably with depth, suggesting either that transport of the high-chloride water occurs as rapidly in the Miami Limestone as in the underlying Fort Thompson Formation or that highly efficient vertical mixing processes occur. Causaras (1987) reports substantial vugular porosity in both formations throughout southern Dade County, and descriptions of cavity-filling detritus are restricted to an upper few feet of rock below surface soils and fill. Therefore, the Miami Limestone permeability might be similar to that of the underlying Fort Thompson Formation below the upper few feet, particularly in southern Dade County where overlying beds of sand are not present.

Results of SASS multiple-well and single-well tests of the Fort Thompson Formation, or in intervals including parts of both the Fort Thompson Formation and Miami Limestone, indicated such high values of transmissivity throughout most of central and southern Dade County that Fish and Stewart (1991, p. 39) merely cite them as greater than 1,000,000 ft^2/d (feet squared per day). Slight errors in estimating well losses were highly significant in the analyses of data from the step-drawdown tests so that more precise estimates of transmissivity could not be made (Rorabaugh, 1953). Drawdowns were negligible in many of the multiple-well tests. Fish and Stewart (1991) indicated an average hydraulic conductivity of the Fort Thompson Formation (and Miami Limestone in some locations) of tens of thousands of feet per day, possibly exceeding an average of 40,000 ft/d.

To the northwest, the Fort Thompson Formation wedges out and has less solution porosity. The line indicating the western limit of the Biscayne aquifer (fig. 3) generally corresponds to a permeable thickness of less than 10 ft. Fish and Stewart (1991) found a few feet of highly permeable limestone of the Fort Thompson Formation at drilling sites on the Tamiami Trail, northwest of this limit. Therefore, permeable material of the Fort Thompson Formation apparently is present everywhere in the study area, although its thickness may be only a few feet in some localities.

Figure 6 shows an area of lower transmissivity in the Biscayne aquifer, extending inland from the coast and ranging from Snapper Creek Canal (C-2) in the north to the mouth of Black Creek Canal in the south. Data indicating lower permeability in the Homestead area from three specific capacity tests of production wells in small well fields are not consistent with results of SASS single-well aquifer tests to the north, south, and west or with other previous tests to the west and east. The only SASS pumping tests in the eastern part of the study area that indicated substantially less permeability than at other central and eastern SASS sites were a test conducted near C-100 and another test conducted farther east near the coast (fig. 6). These latter test results are corroborated by results of several previous tests. Evidence for reduced aquifer permeability in this latter area is accepted as a basis for model construction, but the region of lower permeability probably does not extend southward past the Black Creek Canal.

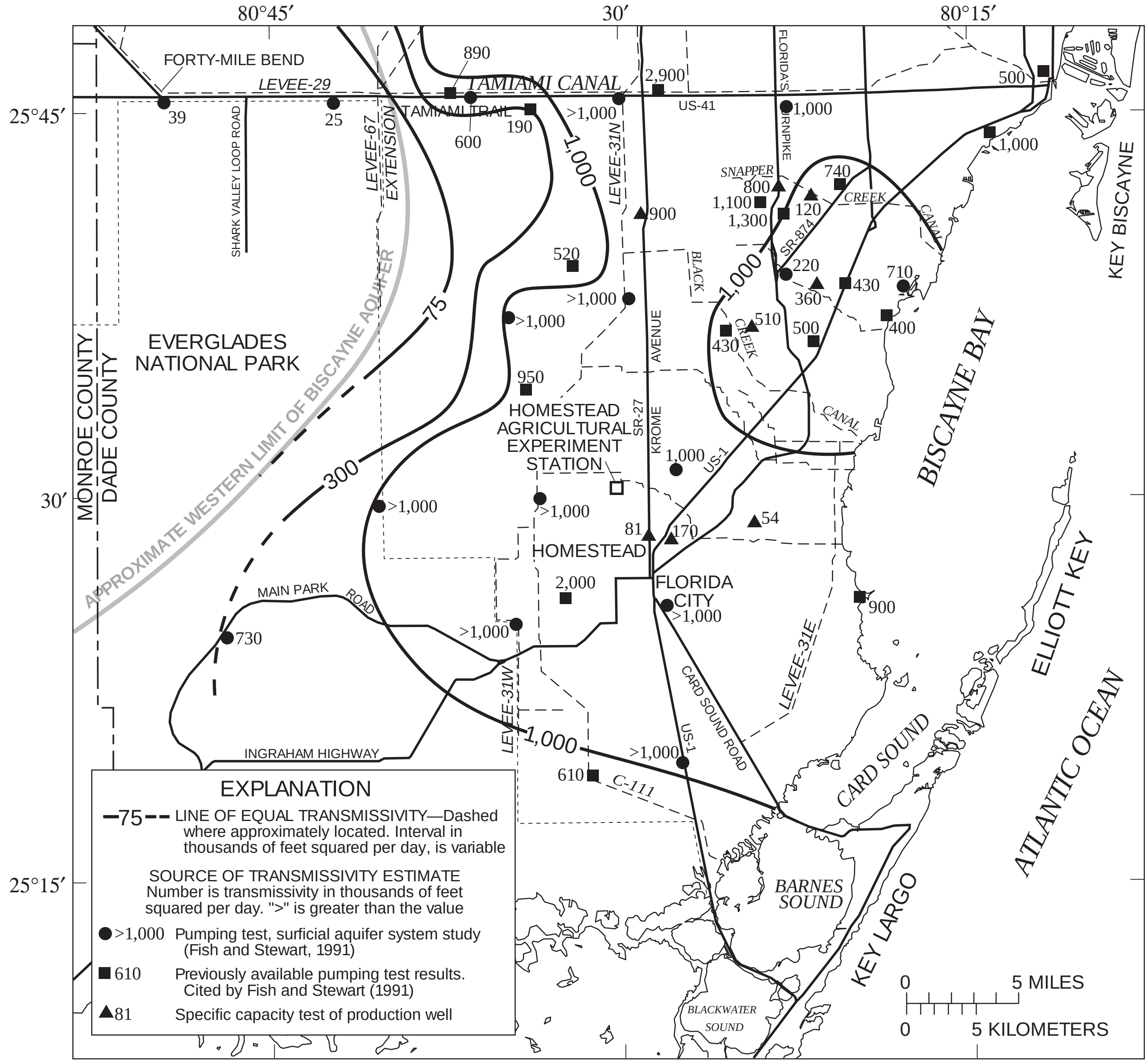


Figure 6. Generalized distribution of transmissivity of the Biscayne aquifer in southern Dade County.

The upper part of the Tamiami Formation contains clastics of very low to moderate permeability throughout the study area, except along the east coast where these deposits are highly permeable. At some SASS sites, the deposits are silty and clayey and are a highly effective confining zone for the underlying gray limestone aquifer. At other SASS sites, the deposits are shelly and of moderate permeability. According to Labowski and others (U.S. Geological Survey, written commun., 1988), the effectiveness of the hydraulic separation of the two aquifers increases westward. Near the eastern boundary of the gray limestone aquifer, however, the confining layer is considered to be leaky.

The permeability of the gray limestone aquifer diminishes toward the southeast as the aquifer thins (Fish and Stewart, 1991). Estimated hydraulic conductivity decreases from 780 ft/d at Forty-Mile Bend in the extreme northwestern part of the study area to between 210 and 410 ft/d at sites to the southeast. Labowski and others (U.S. Geological Survey, written commun., 1988) estimated values of 400 to 500 ft/d in the northeastern part of region 1 (fig. 3), west of L-31N.

Underlying sediments of the Tamiami and Hawthorn Formations are considered by Fish and Stewart (1991) to be of low permeability. Because permeability measurements are lacking, investigators have not been able to make quantitative estimates.

Effective Porosity

Effective porosity is the volume of aquifer pore space through which water and dissolved substances can flow under the influence of prevailing hydraulic gradients. The total porosity of the aquifer is considered to be the sum of solution porosity (secondary porosity) and interstitial porosity (primary porosity), the volume of voids within the rock, or rock pores. Microscopic pathways between rock pores can permit fluid seepage at a slow rate. Where solution features have developed in carbonate rocks, however, effective porosity is identified with the solution porosity because the relative amount of flow through rock pores is negligible compared to that within solution features. If solution features do not exist, flow can only occur through the rock pores, so this porosity is identified as effective porosity.

In unconfined aquifers, specific yield is virtually equal to storage coefficient (Lohman, 1979) because very little water is released from pore compression or retained by pore expansion. In some model codes (INTERA Environmental Consultants, Inc., 1979), the storage coefficient is considered to be a linear function of effective porosity, representing an assumption that it is the effective porosity of surface rocks that is drained by gravity. Thus, effective porosity and specific yield are assumed to be nearly identical in unconfined aquifers with solution porosity, and estimates of one are considered estimates of the other.

Estimates of total or effective porosity of the Biscayne aquifer are rare, and the data collection and aquifer testing of the SASS did not support such estimates. Generally, porosity estimates may be obtained either from direct measurement or indirectly from other measurements. Methods of direct measurement are by laboratory analysis of core samples or by geophysical logging. One indirect method in surficial aquifers is by comparison of rainfall amounts and corresponding water-level changes to obtain specific yield estimates.

Analysis of the porosity of cores provides the most reliable results in rock samples from formations having little or no solution porosity, or from formations in which the solution features are sufficiently small that their spatial distribution is well represented within the volume of the core sample. Neither description applies to the most permeable sections of the Miami Limestone and the Fort Thompson Formation, in which solution cavities are as large as 2 in. in diameter (Fish and Stewart, 1991). The author has examined rock samples from the surficial aquifer system that exhibited fine solution porosity (having only solution channels of small diameter), but laboratory porosity measurements were not made on these samples.

The effective porosity (specific yield) of rocks near the surface can be estimated by comparing the rise of the water table to the intensity of rainfall events. For accuracy, the rainfall should occur in brief intervals and be of sufficiently large amount to render negligible temporal effects of natural drainage and evapotranspiration. It is also assumed that unsaturated zone effects and air compression effects are negligible. Records of rainfall measured daily since 1931 at the Homestead Agricultural Experiment Station (National Oceanic and Atmospheric Administration rainfall station 40910) in southern Dade County (fig. 6) were examined. Daily ground-water levels have also been recorded at the station (USGS local number S-196A) since 1932. The station lies directly between wells G-3314 and G-3315, drilled as part of the SASS, in which highly permeable rocks occur up to land surface (Causaras, 1987; Fish and Stewart, 1991). Only rainfall events before 1966 were used for the analysis because water-level fluctuations might have been strongly influenced by canal drainage since that time. The water level in S-196A remained below land-surface elevation (10.33 ft) during this entire period. The highest measured water level at S-196A in this period was 9.58 ft on October 12, 1947.

Rainfall events were selected in which at least 4 in. of rainfall were concentrated in brief periods preceded and followed by periods of little rainfall. The dates of the rainfall events, the total rainfall, changes in water level, and specific yield estimates (computed as inches of rainfall divided by inches of water-level increase) are listed in table 1. The mean value, omitting 1936 and 1944 estimates that appear anomalous, is 0.225.

Because water-level data might still be affected by some short-term drainage and evapotranspiration, the specific yield estimates (table 1) could be high. Given the variation in the results of individual analyses, the results could support specific yield estimates ranging from 20 to 25 percent for material of the

Table 1. Estimates of specific yield from records of rainfall and the water levels in well S-196A

[Data from Homestead Agricultural Experiment Station, Dade County, Florida]

Number	Date			Rainfall (inches)	Water-level increase (feet)	Specific yield estimate
	Month	Days	Year			
1	October	1-5	1933	13.84	5.25	0.22
2	April	26	1935	4.78	1.55	.26
3	September	2-5	1935	11.15	3.25	.29
4	November	5	1935	5.32	2.25	.20
5	August	17-18	1936	4.52	.50	.75[1]
6	August	21-23	1936	5.54	1.00	.46[1]
7	September	28-30	1937	8.25	3.10	.22
8	June-July	28-30, 1-3	1939	10.15	3.00	.28
9	October	12-19	1939	8.53	4.00	.18
10	May-June	31, 1-2	1940	5.36	2.30	.19
11	April	15-17	1942	8.02	4.00	.17
12	September	8-9	1944	5.10	.85	.51[1]
13	June	10-12	1947	8.78	4.30	.17
14	June	17-18	1947	5.70	1.65	.29
15	August	18-19	1947	4.70	1.90	.21
16	October	11-12	1947	9.71	2.70	.30
17	September	15-19	1948	7.00	1.85	.32
18	September	20-22	1948	8.51	2.15	.33
19	October	4-5	1948	8.75	2.90	.25
20	October	11-12	1949	4.99	1.75	.24
21	October	25	1949	5.92	3.10	.16
22	July	3	1952	4.11	1.40	.25
23	June	20-22	1953	5.43	1.90	.24
24	October	11-13	1955	6.01	2.30	.22
25	May	23-25	1958	12.07	5.20	.19
26	March	18-20	1959	5.61	1.95	.24
27	June	17-22	1959	9.68	3.30	.24
28	November	6-7	1959	4.24	1.80	.19
29	November	19-21	1959	4.83	1.75	.23
30	September	5-10	1960	9.77	3.10	.26
31	September	20-23	1960	6.00	2.50	.20
32	May	26-29	1961	5.64	2.60	.19
33	June	27-28	1961	4.32	1.65	.22
34	June	14-21	1962	10.82	4.80	.19
35	July	9-13	1962	4.57	1.80	.21
36	September	19-25	1963	12.14	3.40	.30
37	June	5-10	1964	6.78	2.70	.21
38	October	27-31	1964	6.59	2.60	.21
39	September	7-9	1965	5.60	3.00	.16
40	October	13-15	1965	4.98	2.10	.20
41	June	1-9	1966	8.91	5.20	.14
Average						.225
Standard Deviation						.047

[1]Anomalous values are not included in average or standard deviation.

Miami Limestone near land surface. Assuming that solution channel development is similar throughout the thickness of the Biscayne aquifer, including Fort Thompson Formation rocks, such values could represent the porosity of the entire thickness of the Biscayne aquifer.

Water Quality

The chemical constituents contained in water of the surficial aquifer system are important to the construction of a regional flow model to the extent that these chemicals affect the flow system. Ground-water flow can be affected by density variations, if the variations are sufficiently large. Water density varies with pressure, temperature, and dissolved-solids concentration (salinity). Of these, only salinity shows sufficient spatial variation in the study area to have an appreciable effect on fluid density.

Specific conductance, which correlates closely with dissolved-solids concentration, and chloride concentrations were measured at sequential depths during drilling of test wells for the SASS, and these data were later supplemented by water samples collected from monitor wells drilled to various depths (Sonntag, 1987). The dual-tube reverse-air drilling method that was used generally made it possible to collect samples representative of the aquifer below the depth of the outer drilling tube. Analyses of the drilling water samples and of those from monitor wells correlated closely. The few exceptions were probably attributable to seasonal movement of coastal saltwater in highly permeable zones. The only substantial areal variations in specific conductance were near the coast. Inland, 500 μS/cm (microsiemens per centimeter at 25 degrees Celsius) is a representative value for the average specific conductance of ground water, and 200 to 400 mg/L (milligrams per liter) and 10 to 50 mg/L, respectively, are representative ranges of values for dissolved-solids and chloride concentrations.

According to Sonntag (1987), the vertical specific conductance profile at a well less than 1 mi from the Atlantic Ocean in the northeastern part of the study area where hydraulic gradients are relatively steep shows no evidence of saltwater intrusion to a depth of 215 ft. Farther south along the east coast where hydraulic gradients are lower, saltwater was detected in a well located 2 to 3 mi inland from the coast of Biscayne Bay. In this coastal area, freshwater flows are probably restricted to an upper lens that thins seaward.

In the southern glades (region 3), where hydraulic gradients are lowest and decrease gradually toward the southern coast at Florida Bay (fig. 3), saltwater intrusion is more areally extensive. Two SASS wells drilled 15 mi inland from the coast of Florida Bay had saline water (more than 1,500 mg/L of dissolved-solids concentration) at elevations near the base of the surficial aquifer system, though Biscayne aquifer water was fresh. Virtually no freshwater was found in one SASS well drilled 4.5 mi inland. Data from two SASS wells in the southeastern coastal part of the study area, about 6 to 7 mi from the coasts of Barnes Sound and Florida Bay, indicated a pattern of freshwater flowing over a lens of highly saline water (about 23,000 mg/L dissolved solids) in the lower part of the Biscayne aquifer. The highly saline water was underlain by much less saline water (600 to 1,400 mg/L dissolved solids) in less permeable materials of the Tamiami Formation.

Water-Table Altitude

The simulation of aquifer heads, their seasonal fluctuations, and the regional pattern of ground-water flow is the principal focus of many subsequent parts of this report. In this section, a brief qualitative discussion and comparison of measured water levels and characteristics of their seasonal fluctuations in selected wells in the seven soil subregions (fig. 3) is presented to elucidate features of the simulation problem. The comparison is of interest because the different soil types correlate well with different sets of hydrologic processes. The period analyzed comprises water years 1974 to 1978, and hydrographs from that period are shown in figure 7. A list of the selected wells and the other ground-water wells and surface-water stations used in this report is given in table 2.

Well G-620 is open from 6 to 16 ft below land surface slightly northwest of the center of Shark River Slough, which generally corresponds to region 1B (Everglades peat) in figure 3. The depth interval corresponds to a thin lens of Miami Limestone and Fort Thompson Formation of unknown permeability. The water level (fig. 7) usually is at or above land surface during the annual wet season (June-October) and for several months thereafter. During the dry spring months (March-May) of these water years, the water table falls below the surface of the thin peat layer and drops precipitously because of evapotranspiration and perhaps some drainage by ground-water flow. The ground-water recession pattern is typical of the Everglades peat region and unlike that observed in coastal ridge wells.

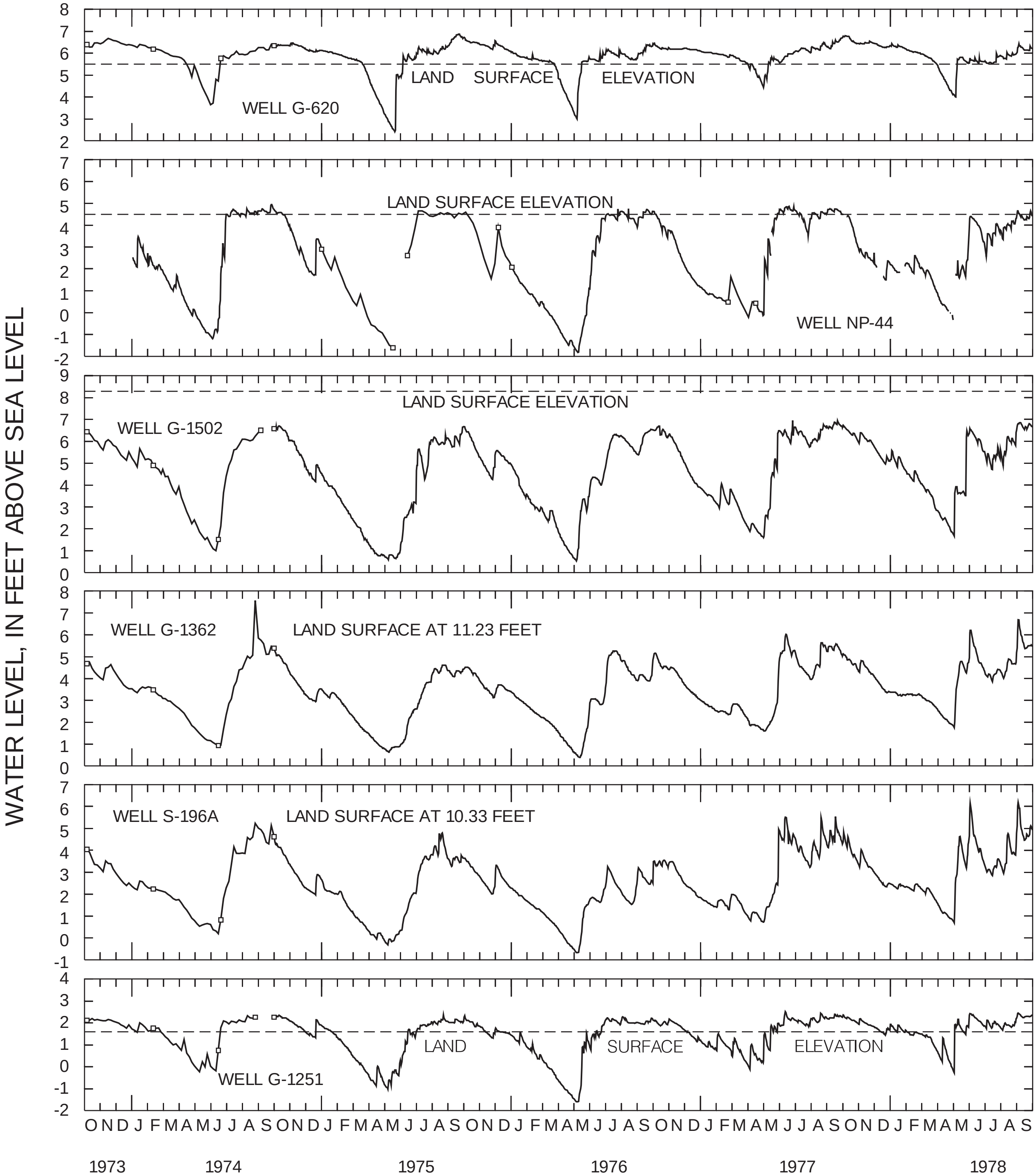


Figure 7. Water-level fluctuations in selected wells in southern Dade County for water years 1974-78.

Wells NP-44 and G-1502 are in locations representative of the water table of the rocky glades (fig. 3, region 2A). NP-44, 32 ft deep and cased to 10 ft, is near the southwestern extremity of region 3 where the Biscayne aquifer is about 30 ft thick. Well G-1502, 31 ft deep and cased to 11 ft, is in the central part of the rocky glades (region 2A) very near the flowing Grossman well in Chekika State Recreation Area (fig. 2). Wet-season water levels (fig. 7) are close to land-surface elevation in both wells and show

Table 2. Summary of ground- and surface-water stations used for regional flow-model construction and calibration[1]

[Period of record: 1990+ indicates that data collection was continuing at the time of the first draft of this report (1990). Agency: USGS, U.S. Geological Survey; NPS, National Park Service. Additional notations: e, estimated value; —, value is unknown or not estimated]

USGS numerical designation	Local number or name	Map identifier	Period of record	Data available in USGS Miami data base	Land[2] surface datum (feet above sea level)	Agency	Use
253029080295601	S-196A		10/32-1990+	All	10.33	USGS	Calibration
None	Tamiami Canal at Bridge 61, Krome Avenue[3]	TCBR61	11/39-03/54	None	6.3e	USGS	Boundary
02288990	Tamiami Canal at Forty-Mile Bend	TC40MB	12/39-10/80	All	7.3e	USGS	Boundary
253549080214101	S-182A		01/40-1990+	All	11.14	USGS	Calibration
02288995 02289060	Tamiami Canal at Bridge 45[4]	TCBR45	01/40-08/69	All	6.2e	USGS	Boundary
02289500	Tamiami Canal near Coral Gables	TCCGBL	01/40-1990+	After 10/59	—	USGS	Boundary
252829080285101	F-358		02/40-1990+	All	7.76	USGS	Calibration
254605080205901	G-10		03/40-05/72	All	—	USGS	Boundary
254332080200802 254332080200801	G-39		05/40-08/76	All	8.08	USGS	Boundary
254000080000000	Tamiami Canal, west of Dade-Broward Levee[5]	TCDBLV	08/41-12/55	All	6.3e	USGS	Boundary
254000080000001	Tamiami Canal, east of Dade-Broward Levee[5]	TCDBLV	07/42-12/55	All	6.3e	USGS	Boundary
02290732	Biscayne Bay near Homestead	TDLHMS	02/46-09/81	After 10/62	—	USGS	Boundary
253902080202501	G-553		07/47-1990+	All	12.11	USGS	Boundary
253212080221901	G-518		10/47-05/58	All	5.65	USGS	Calibration
254130080234501	G-551		12/47-01/60 03/85-1990+	All	8.04	USGS	Calibration
254207080200301	G-595A		07/49-09/83	All	8.90	USGS	Boundary
253937080304001	G-596		07/49-1990+	All	7.28	USGS	Calibration
252717080251701	G-612		05/50-07/57	All	3.90	USGS	Calibration
252425080320001	G-613		05/50-1990+	All	6.06	USGS	Calibration
253258080264301	G-614		05/50-1990+	All	11.10	USGS	Calibration
254600080350001 254500080360001	G-618		11/50-1990+	All	7.40/6.35	USGS	Boundary
254540080455201	G-619		11/50-06/66	All	7.20	USGS	Boundary
254000080460001	G-620		11/50-1990+	All	6.13/5.5	USGS	Calibration
02290815	P-33		10/52-10/80 11/80-1990+	All None	5.2	USGS NPS	Calibration
02290820	P-38		10/52-10/80 10/80-1990+	All None	.9	USGS NPS	Boundary

Table 2. Summary of ground- and surface-water stations used for regional flow-model construction and calibration[1] —Continued

[Period of record: 1990+ indicates that data collection was continuing at the time of the first draft of this report (1990). Agency: USGS, U.S. Geological Survey; NPS, National Park Service. Additional notations: e, estimated value; —, value is unknown or not estimated]

USGS numerical designation	Local number or name	Map identifier	Period of record	Data available in USGS Miami data base	Land[2] surface datum (feet above sea level)	Agency	Use
02290870	P-34		02/53-10/80 11/80-1990+	All None	1.3	USGS NPS	Boundary
02290810	P-37 (also NP-207)		02/53-10/80 10/80-1990+	All None	.6	USGS NPS	Calibration
02290830	P-35		03/53-09/80 10/80-1990+	All None	.9	USGS NPS	Boundary
02289090	Tamiami Canal, west of L-30[6]	TCL30W	03/54-09/80	03/54-09/74	—	USGS	Boundary
02289250	Tamiami Canal, east of L-30[6]	TCL30E	03/54-09/80	03/54-09/74	—	USGS	Boundary
253537080284401	G-757A		04/56-1990+	All	9.06	USGS	Calibration
252928080332401	G-789		04/56-1990+	All	6.70	USGS	Calibration
254038080280201	G-855		07/58-1990+	All	7.90	USGS	Calibration
02290540	Biscayne Bay at Coconut Grove	TDLCGR	02/59-09/81	After 10/62	--	USGS	Other
253854080242801	G-858		04/59-1990+	All	8.55	USGS	Calibration
253718080192301	G-860		04/59-1990+	All	8.28	USGS	Boundary
253345080342301	G-863A		04/59-10/61	All	8.88	USGS	Calibration
252612080300701	G-864		04/59-1990+	All	8.87	USGS	Calibration
252553080431800	NP-44		04/60-12/70 01/69-05/77 05/77-09/80 10/80-1990+	All None All None	5.86/5.0	USGS NPS USGS NPS	Calibration
251910080474601	NP-46		04/60-06/71 05/77-10/80 10/80-1990+	All All None	3.11/1.0	USGS USGS NPS	Calibration
02290825	Florida Bay at Flamingo	TDLFLM	08/60-10/80	After 10/62	—	USGS	Boundary
02290800	Taylor Slough near Homestead	TSHMST	08/60-10/80 10/80-1990+	All None	—	USGS NPS	Calibration
253900080343001	G-861		10/61-11/69	All	6.42	USGS	Calibration
252918080234201	G-1183		11/61-1990+	All	6.17	USGS	Calibration
253345080342301	G-863		12/61-09/69	All	6.87	USGS	Calibration
251950080390201	NP-67		06/62-12/68 01/69-06/71 06/76-09/76 10/76-09/80 10/80-1990+	All None None All None	4.88/2.0	USGS NPS NPS USGS NPS	Calibration

Table 2. Summary of ground- and surface-water stations used for regional flow-model construction and calibration[1] —Continued

[Period of record: 1990+ indicates that data collection was continuing at the time of the first draft of this report (1990). Agency: USGS, U.S. Geological Survey; NPS, National Park Service. Additional notations: e, estimated value; —, value is unknown or not estimated]

USGS numerical designation	Local number or name	Map identifier	Period of record	Data available in USGS Miami data base	Land[2] surface datum (feet above sea level)	Agency	Use
252345080421201	NP-72		06/62-01/68 01/68-07/76 05/77-09/80 10/80-1990+	All None All None	4.90	USGS NPS USGS NPS	Calibration
252622080470201	NP-62		10/62-12/68 01/69-07/76 05/77-10/80 10/80-1990+	All None All None	4.48/2.0	USGS NPS USGS NPS	Calibration
02289041	Tamiami Canal below S-12C	TCS12S	04/63-1990+	All	1.5e	USGS	Bundary
251922080340701	G-1251		04/65-1990+	All	1.75e	USGS	Calibration
02290750	Card Sound at Model Land Canal	TDLMLC	02/67-01/82	All	—	USGS	Bundary
02290803	Taylor Slough at Royal Palm	TSRPLM	02/68-09/80	All	—	USGS	Calibration
02290828	P-36		02/68-09/80 10/80-1990+	All None	2.8	USGS NPS	Calibration
02290769	C-111 above S-18C	S-18C	10/68-1990+	All	1.5e	USGS	Calibration
263630080264801 263630080264801	G-1362		12/68-1990+	All	11.23	USGS	Calibration
243233080301001	G-1363		12/68-1990+	All	9.78	USGS	Calibration
02289060	Tamiami Canal at Bridge 53 (published as Tamiami Canal Outlets), L-30 to L-67A	TCBR53	08/69-1990+	All	6.4e	USGS	Calibration
253012080261401	G-1486		04/70-1990+	All	11.54	USGS	Calibration
254054080295401	G-1487		04/70-1990+	All	7.52	USGS	Calibration
253656080350301 252656080350301	G-1502		07/70-1990+	All	8.28	USGS	Calibration
02290827	L-67 Extended Canal near Richmond	L67XRD	06/71-10/80 01/81-1990+	All None	6.4e	USGS NPS	Calibration
02290829	NP-204		10/73-05/80	All	2.0e	USGS	Calibration
02290832	NP-203		10/73-05/80 06/80-1990+	All None	5.2e	USGS NPS	Calibration
02290811	NP-206		10/74-05/80 01/81-1990+	All None	5.2e	USGS NPS	Calibration
02290861	NP-201		10/74-05/80 11/83-1990+	All None	6.8e	USGS NPS	Calibration

Table 2. Summary of ground- and surface-water stations used for regional flow-model construction and calibration[1] —Continued

[Period of record: 1990+ indicates that data collection was continuing at the time of the first draft of this report (1990). Agency: USGS, U.S. Geological Survey; NPS, National Park Service. Additional notations: e, estimated value; —, value is unknown or not estimated]

USGS numerical designation	Local number or name	Map identifier	Period of record	Data available in USGS Miami data base	Land[2] surface datum (feet above sea level)	Agency	Use
02290868	NP-205		10/74-05/80 05/80-1990+	All None	—	USGS NPS	Other
02290862	NP-202		01/75-05/80 01/81-1990+	All None	5.5e	USGS NPS	Calibration
253735080402100	L-67 Extended Canal at south end near Coopertown	L67XSE	06/76-09/80	All	6.0e	USGS	Calibration
254130080380500	Northeast Shark River Slough no. 1	NESRS-1	07/76-10/80 07/82-1990+	All	6.0e	USGS	Calibration
254315080331500	Northeast Shark River Slough no. 2	NESRS-2	07/76-10/80 04/82-1990+	All	5.8e	USGS	Calibration
254754080344300	Shark River Slough no. 1	SRS-1	08/76-10/80 09/82-1990+	All	—	USGS	Other
251148080410300	Taylor Slough at Craighead Lake (Pond)	TSCRLK	10/78-09/80 10/80-1990+	All None	.6e	USGS NPS	Other
251946080254800	Everglades no. 1	EVR-1	04/84-1990+	All	1.5e	USGS	Calibration
251855080283401	G-3354		09/85-1990+	All	1.36	USGS	Calibration
251855080283400	Everglades no. 2B	EVR-2B	09/85-1990+	All	1.36	USGS	Other
252502080253901	G-3356		10/85-1990+	All	2.5e	USGS	Calibration

[1]Ground-water stations have daily values, except for water years 1958-73 for which 5-day values are available. Surface-water stations have daily values, unless otherwise noted. Table is arranged chronologically according to the date of the first available data.

[2]Where two values are given for a specific station, the first number represents land-surface datum and the second number represents estimated average local land-surface elevation. The station datum may be unrepresentative of average local conditions because wells in wetland areas are drilled on elevated roadbeds or surrounded by berms to provide all-season access.

[3]Daily values to 07/42; periodic afterward.

[4]Periodic readings to 02/52; after 10/63, published as Tamiami Canal Outlets, L-30 to L-67A.

[5]Daily values to 01/46; periodic afterward.

[6]Periodic staff gage readings only (generally at least one per month).

relatively small variation because of the influence of nearby surface water (overland flow, which has a high specific yield (1.0) and a more slowly changing head than the subsurface water table in noninundated areas.

In well NP-44 and other nearby wells, the dry-season recession is highly pronounced in comparison with observed recessions in other parts of southern Dade County. Recessions are less pronounced in well G-1502 and less still in wells S-196A, 20 ft deep (casing depth unknown), and G-1362, 33 ft deep and cased to 11 feet, which are centrally located in the coastal ridge region of relatively high land elevation (fig. 3, region 2B). Dry-season recessions are dampened even further in well G-1251, 59 ft deep and cased to 5 ft below land surface, in the southern coastal plain (fig. 3, region 3A). Here, wet-season water levels are relatively stable, remaining near land surface.

To provide numerical examples, the dry-season recession from December 1, 1975, to May 1, 1976, was 5.75 ft at NP-44, 5.0 ft at G-1502, 3.75 ft at S-196A, and 3.25 ft at both G-1362 and G-1251. The comparison is made in a time period when the NP-44 water level remained below land surface and did not reflect local surface-water stage during flooding. These differences are significant in the hydrologic regime of Dade County and are typical of regional variations.

The regional differentiation of dry-season recessions is related to regional variations in the rate of surface-water and aquifer drainage and its relation to recharge and evapotranspiration rates. These differences must be replicated for a computer simulation of the flow system to be accurate, as must be the relation between ground-water levels and surface-water stages during the wet season in areas near flowing surface water.

Natural Surface-Water Flow Systems

Natural surface-water flows occur in Dade County south of the Tamiami Trail as: (1) overland sheetflow in inland areas of lower-than-average land elevation with poor infiltration into underlying ground-water flow systems; or (2) small, ill-defined episodic or perennial streams draining the southern and eastern glades. The mainland is also bounded on the east and south by broad, shallow saltwater embayments, Biscayne Bay and Florida Bay, which open to the Atlantic Ocean. Although these water bodies do not seasonally invade the land area, the local and seasonal variation of stage in the embayments that receive discharges of ground-water and surface-water flow from southern Dade County is of considerable significance for attempts to simulate the flow system.

Seasonally Inundated Wetlands

Soils regions 1A, 1B, and 2A (fig. 3) underlie a segment of the Everglades, which intense summer rains transform into a vast, shallow surface-water flow system that has been likened to a river of grass (Douglas, 1947). The Indian term for it was Pah-Hay-okee (“grassy water”). Before the construction of roads, canals, and control structures, surface water during the wet season flowed southward across the northern boundary of the area, curved westward, and exited across the western boundary toward an area of mangrove swamps draining into the coastal estuaries of southwestern mainland Monroe County by the Shark and Harney Rivers (fig. 1). The direction of flow in the Everglades is illustrated by the tree islands shown in figures 8 and 9. Gentle scouring and abrasion by water flow over long periods have caused these “islands,” which rise 0.5 to 2 ft above surrounding land surface and are covered with hardwoods, to become elongated in the direction of surface flow (Parker and others, 1955, p. 152).

The center of the flow system, Shark River Slough, is a northeast-southwest oriented valley where land surface is 1 to 3 ft below the surface of land to the northwest and southeast. The valley closely corresponds to soil region 1B. The width of the surficial flow system and the depth of inundation show marked seasonal variation. The approximate extent of lands subject to inundation in the study area during typical wet and dry seasons is shown in figures 8 and 9. These maps are generalizations based on long-term hydrographs from surface-water stage and ground-water elevation stations throughout the study area. They are not based on a concentrated distribution of field measurements throughout the areas shown and, therefore, should not be used for detailed interpretations.

The degree of inundation depends on the intensity and duration of wet-season precipitation. Based on available data, there does not seem to be any part of the land mass within the study area that normally remains permanently inundated. Even at station P-35 in the lower part of Shark River Slough, the surface-water stage normally drops to land surface, or a few inches below in measuring pits, late in the dry season.

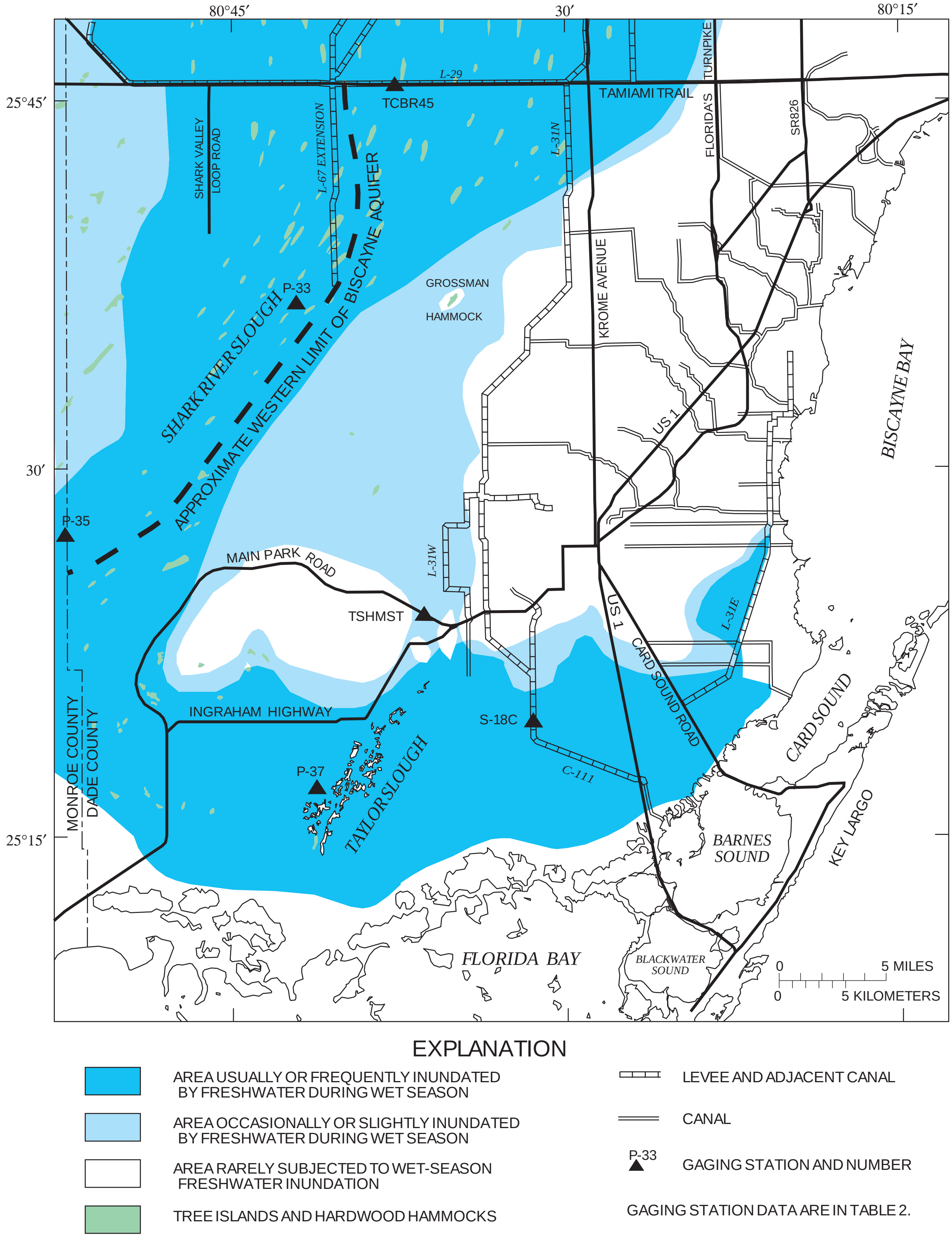


Figure 8. Areas subject to frequent or occasional wet-season freshwater inundation in southern Dade County.

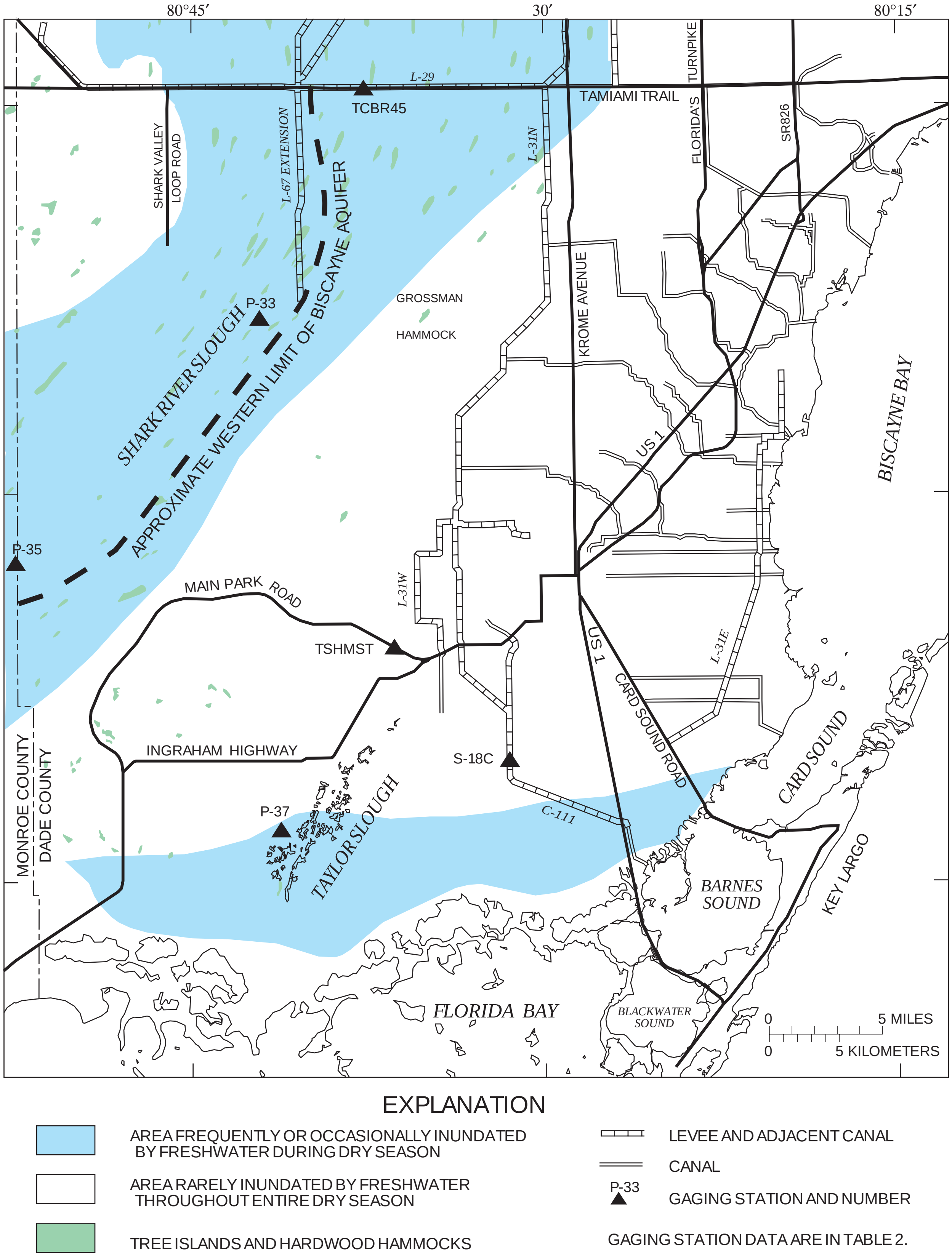


Figure 9. Areas occasionally subject to freshwater inundation throughout entire dry season in southern Dade County.

Surface water in the slough during the dry season is locally impounded in grassy ponds and "alligator holes," pits dug by these reptiles to provide themselves with an aquatic environment until wet-season rains recur. Figure 9 shows regions where a slight degree of inundation may persist throughout dry seasons of less than normal severity.

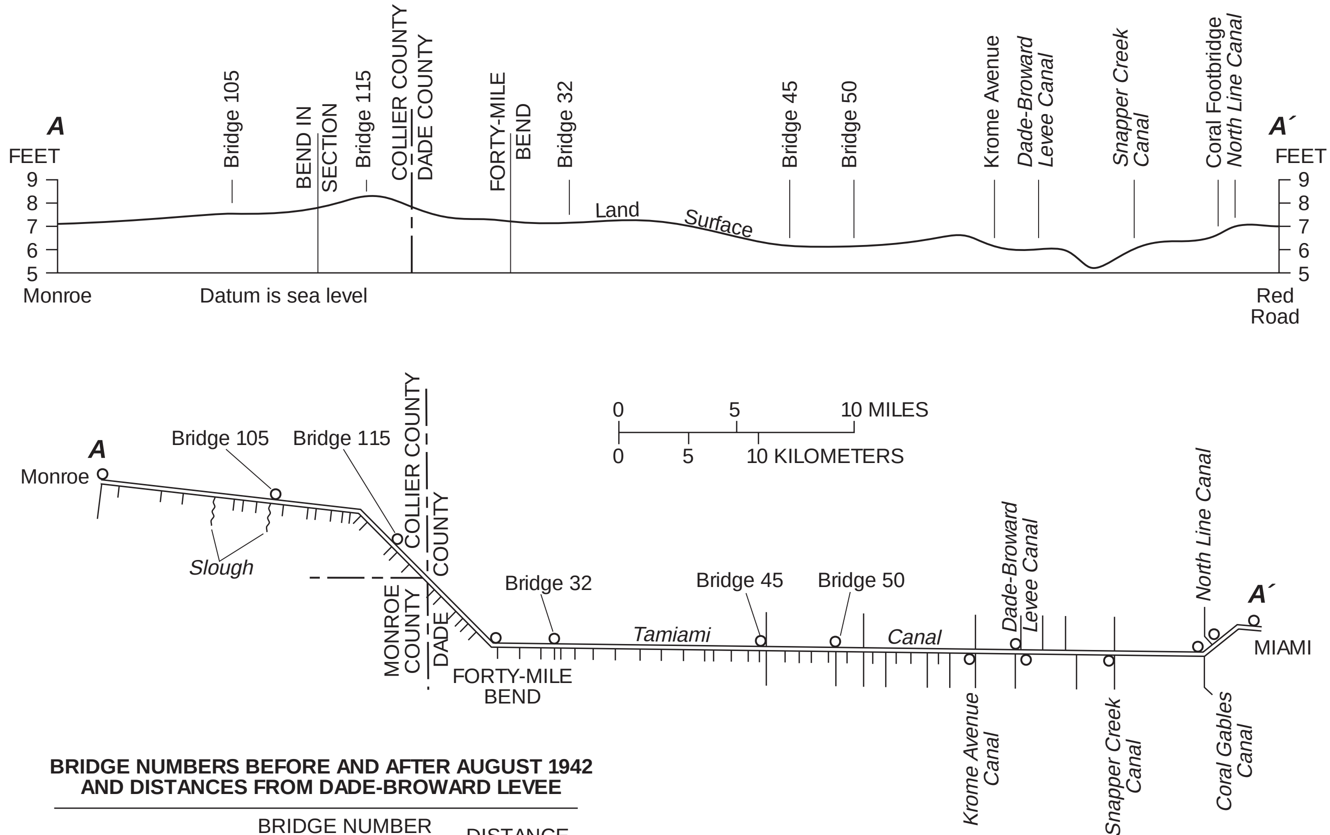
The series of open lakes and ponds that extend southwestward in the south-central part of the study area are part of a region of lower elevation (Taylor Slough) that channels surface water to the coastal southern glades. Although figures 8 and 9 do not show it, Taylor Slough extends farther north as a broad, shallow channel for episodic surface flow during high rainfall periods. The channel lies in an area shown in figure 3 as a northward incursion of calcitic mud (region 3A) into the rocky glades region (fig. 3, region 2A). Land elevations in this area are 1 to 3 ft below those of the surrounding land. The region of lower elevation continues northeastward of the marl region, ending just west of Grossman Hammock, location of the Grossman well (fig. 2), an area surrounding well G-1502 in figure 3. (Grossman Hammock is also shown in figure 8 as hardwood hammock surrounded by an elliptically shaped area of normally noninundated land.) The upper and lower sections of Taylor Slough are connected through the westward extension of the coastal ridge into Everglades National Park (fig. 3, Long Pine Key) by several channels for wet-season flows, shown in figure 3 as continuous sections of surficial calcitic mud deposits near the lower end of L-31W. Even the most southern part of Taylor Slough usually becomes dry during the annual dry season.

Where surface water flows over peat soils, the water encounters resistance to flow from dense vegetation. In addition, land-surface elevation varies locally, causing a local clustering of different vegetation and animal habitats (Kolipinski and Higer, 1969). The average depth of flowing surface water tends to be relatively uniform throughout Shark River Slough (figs. 3, 8, and 9) and rarely exceeds 2 ft, so that variations in land-surface elevation contribute to the resistance to flow. Surface flow in the southern glades is also impeded by vegetation and rarely exceeds 1.5 ft in depth. Where shallower surface-water flows occur in the rocky glades region, the irregularity and roughness of the limestone surface and the presence of shallow solution pockets filled with limey soil, in which various plant species grow densely, tend to impede flow.

Near Florida Bay along the southern edge of region 3B (fig. 3) occurs a natural hydrologic feature referred to as the Buttonwood Embankment by Craighead (1971). The growth of mangrove forests along the southern fringe of land has caused the buildup of a layer of muck less than 1 ft thick, penetrated only by scattered streams. During the dry season, the embankment impounds freshwater flows from the north, as well as emergent ground water, so that a few inches of surface water remain even when ground-water levels farther north decrease below sea level because of evapotranspiration and lack of precipitation.

A typical day during a relatively intense wet period before construction of most of the levee system in Shark River Slough, September 15, 1959, is selected to illustrate stage and depth-of-inundation relations in the Everglades and southern wetlands. Tamiami Trail transected the slough on this date, but southward surface flow past the road remained relatively unimpeded by virtue of culverts underneath the numerous bridges (fig. 10). The Tamiami Trail borrow canal north of the road served as a collector canal that connected the series of culverts. Figure 10 does not show L-31N, constructed in 1952, which extended southward from the road about 1 mi west of Krome Avenue.

The recorded stage at Bridge 45 (TCBR45 in figs. 8-9; Bridge 45 in fig. 10), approximately where Tamiami Trail crosses the center of Shark River Slough, was 8.76 ft on September 15, 1959. The average land-surface elevation at the site is 6.2 ft. At station P-33 (figs. 8 and 9), about 11.6 mi down the curving axis of the slough, surface water with a stage of 6.96 ft flowed over a land surface having an elevation of about 5.2 ft. This represents a stage reduction of about 0.16 ft/mi (foot per mile) and an average land-surface slope of about 0.09 ft/mi. At station P-35 (figs. 8 and 9), about 15 mi farther down the slough and at the head of the river-estuary system of the Shark and Harney Rivers, the surface-water stage was 2.75 ft. This represents a larger stage reduction of about 0.28 ft/mi between the locations of P-33 and P-35. The average land-surface elevation is estimated to be 0.9 ft at P-35; therefore, the average land-surface slope between the sites is also larger (0.29 ft/mi). At station P-37 (figs. 8 and 9) in the lower part of Taylor Slough, the surface-water stage was 2.19 ft, and land-surface elevation is about 0.6 ft.



BRIDGE NUMBERS BEFORE AND AFTER AUGUST 1942 AND DISTANCES FROM DADE-BROWARD LEVEE

LOCATION	BRIDGE NUMBER Before August 1942	BRIDGE NUMBER After August 1942	DISTANCE (miles)
Dade-Broward Levee	186	62	0.0
Krome Avenue	185	61	2.0
	174	50	8.25
	169	45	11.1
	156	32	20.85
Forty-Mile Bend	—	—	23.5
Dade-Collier Line	—	—	27.55
	143	115	29.6
	133	105	35.55
Monroe	124	96	43.05

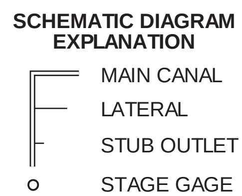


Figure 10. Tamiami Canal bridges, outlets, and gaging stations before 1962 and land-surface elevation along the canal.

Hydrographs showing stages at five Shark River Slough, Taylor Slough, and southern glades surface-water stations are presented in figure 11 for water years 1974-78, the same period that was used to illustrate water-table variations (fig. 7). Most wetlands surface-water stations are designed (some are in holes blasted out with explosives) to continue measuring stages when they are below land surface.

Stations P-33 and P-35 in Shark River Slough rarely had stages below land surface during water years 1974-78. When stage is below land surface, as at P-33 near the eastern border of the slough in 1975 (fig. 11), the subsequent ground-water recession is very rapid, as was also observed at well G-620 (fig. 7) near the western border of the slough. Land surface is slightly higher at G-620, and dry periods of significant duration occur each year during the period illustrated. Peats in Shark River Slough remain moist during long dry periods when the water table is below land surface, and the surface seems soggy when trodden upon.

The station designated Taylor Slough near Homestead (figs. 8 and 9, TSHMST), located where Taylor Slough bisects the coastal ridge, is north of the regularly inundated section of the slough. The land-surface elevation at the station is less than 5 ft. Surrounded by higher elevation land, the water table at this station shows large seasonal variations that seem more characteristic of ground-water heads in noninundated areas (fig. 7).

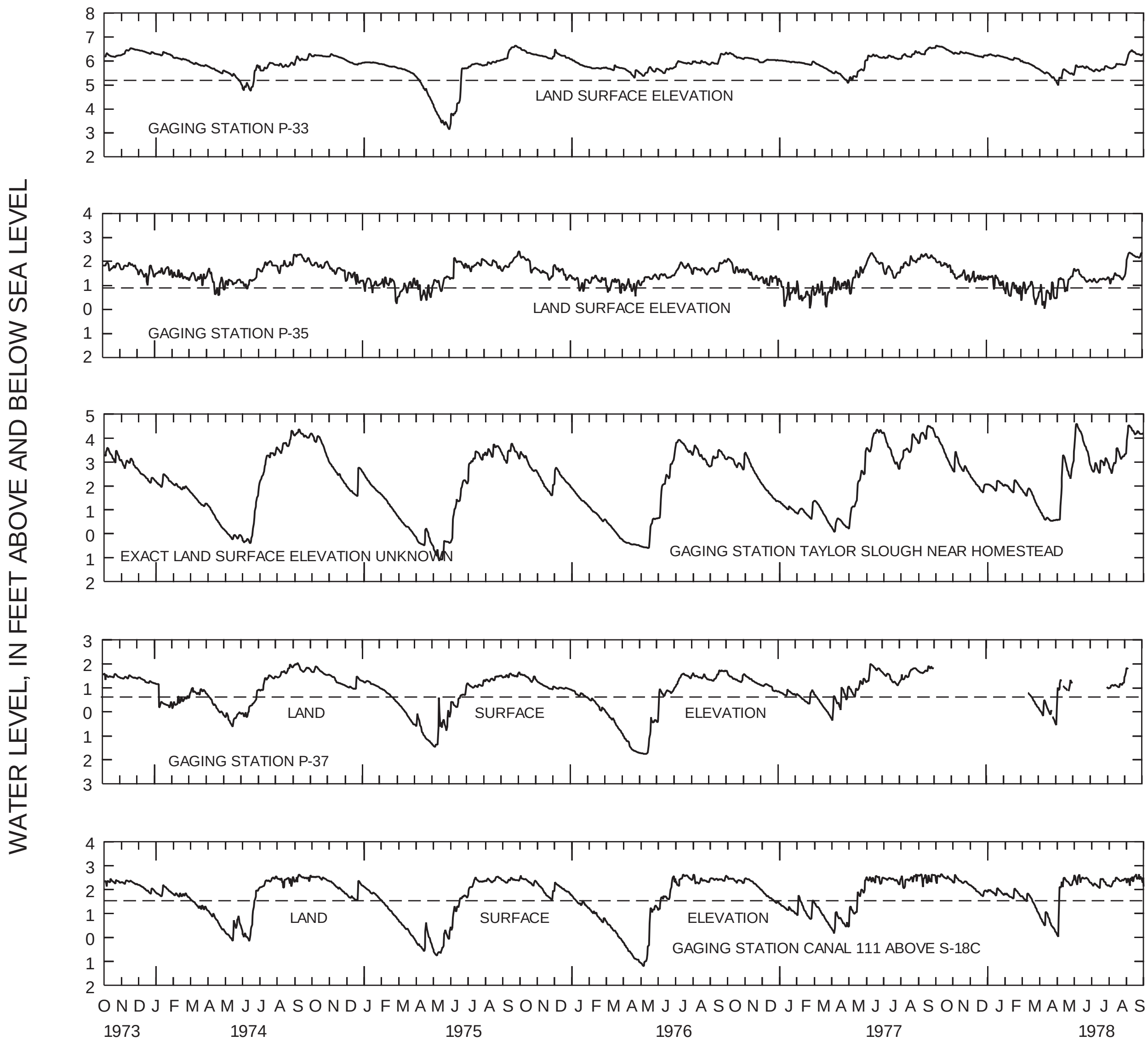


Figure 11. Stage fluctuations at selected gaging stations in southern Dade County for water years 1974-78.

The stage record at P-37, at the western edge of the seasonally inundated section of Taylor Slough, indicates the occurrences of long dry periods characterized by rapid ground-water recessions. The record from the station designated C-111 above S-18C (figs. 8 and 9, S-18C), located in the southern glades farther to the east, indicates similar stage behavior.

Attempts by the USGS to measure the velocity of surface flows in Shark River Slough have been unsuccessful because of the high degree of dispersion caused by varying wind velocity. Estimates of surface-water flow rates as high as 1,500 ft/d were made by Leach and others (1972, p. 26) based on estimates of surface flow through culverts on the Tamiami Trail, subject to the assumption that all flows through the culverts entered the surface-flow system.

Transverse Glades

Much of the coastal ridge is appreciably higher in elevation (10-16 ft) than the 5- to 9-ft elevation of the rocky glades farther inland or the 0- to 3-ft elevation of the coastal glades to the east. However, the coastal ridge is transected by channels of lower elevation that divide the ridge into high elevation "islands." These channels are covered with mucky soils and, before local urbanization and intensive water management, were occasionally inundated during wet periods. Hence, the term "transverse glades" has been applied to these northwest-southeast oriented features (partially shown in fig. 3 as narrow sections of surface calcitic mud, extending northwestward across the coastal ridge). Land-surface elevations within each of the transverse glades decrease monotonically coastward from 8 to 9 ft at their upstream ends in the eastern rocky glades.

A general description of the transverse glades was provided by Parker and others (1955, p. 146), and the glades are discussed in detail by Hartwell (1970). Documentation of the precise flow characteristics of the transverse glades seems to be virtually nonexistent. Long-term residents of the area recall the southeastward sections of some of these channels to be filled with flowing water for substantial periods of time during wet seasons. As suggested by Parker and others (1955, p. 333), water may have flowed from the rocky glades to the coast during extreme high water, as might occur following tropical storms. In normal wet seasons, the lower reaches might have collected runoff and ground-water seepage.

Flow in the transverse glades diminished substantially after the water table in central Dade County was lowered by construction of Black Creek Canal and L-29 (fig. 6) in 1962. Construction of a network of drainage canals from 1962 to 1968 was facilitated by locating them in these low-elevation channels (fig. 3). To take advantage of the rich soil, other parts of the transverse glades in southern Dade County have recently been used for growing winter crops or nursery plants. Increasingly, housing developments are being located in the remaining transverse glades. Typically, several feet of fill are placed at each unit location to raise the foundation level for flood protection.

Coastal Embayments

The broad embayments that border the eastern and southern coasts of Dade County (fig. 12) do not exceed 10 ft in depth and are sheltered from the Atlantic Ocean by a chain of barrier islands, or “keys,” separated from each other by shallow channels. The Florida Keys lie on the eastern and southern boundary of the Floridan Plateau (Uchupi, 1966), beyond which the ocean bottom drops abruptly to as much as 2,500 ft below sea level. The largest embayon the south, are separated by smaller embayments. Card Sound, an inlet of Biscayne Bay, is linked to Barnes Sound by a large natural channel. Barnes Sound is linked to Florida Bay through Blackwater Sound by sections of the Intracoastal Waterway that are narrow dredged cuts bisecting natural causeways.

The Biscayne aquifer extends eastward and southward beneath the embayments to the edge of the Floridan Plateau. In this region, little or no data describing aquifer characteristics have been acquired. Seaward-flowing freshwater in the Biscayne aquifer may discharge by upward seepage to surface water in the eastern and southern glades, as suggested by the shallow depth to saltwater in wells in the southern glades, or into the northern and central parts of Biscayne Bay. According to Parker and others (1955, p. 580-584), natural springs discharged substantial amounts of freshwater to Biscayne Bay before 1910, when construction of the drainage system in southern peninsular Florida began. Data are not available to show conclusively whether freshwater discharges to the bay presently occur, though variations in bottom vegetation have been correlated with variations in specific conductance possibly caused by ground-water discharge (Kohout and Kolipinski, 1967), and a head gradient has been shown to exist from the aquifer underlying the bay 200 ft offshore (Kohout, 1967). However, wherever discharge now occurs, the governing hydraulic influence is the head of the coastal seawater. For this reason, data were analyzed to determine the spatial distribution and annual periodicity of stages in the coastal embayments.

The locations of four sites where daily tidal stages were recorded for more than 10 years are shown in figure 12. The sites are Florida Bay at Flamingo (TDLFLM), Card Sound at Model Land Canal (TDLMLC), Biscayne Bay near Homestead (TDLHMS), and Biscayne Bay at Coconut Grove (TDLCGR). Data from each site were entered into the USGS computer files in various forms as described below:

Data format	TDLFLM (water years)	TDLMLC[1] (water years)	TDLHMS (water years)	TDLCGR (water years)
Daily average	1963-65	--	1963-66	1963-65
Daily high and low	1966-73	1967-74	1967-81	1966-81
Low high, high, high low, low (each day)	1974-80	1974-80	--	--

[1]Data are available for only part of the first and last water years.

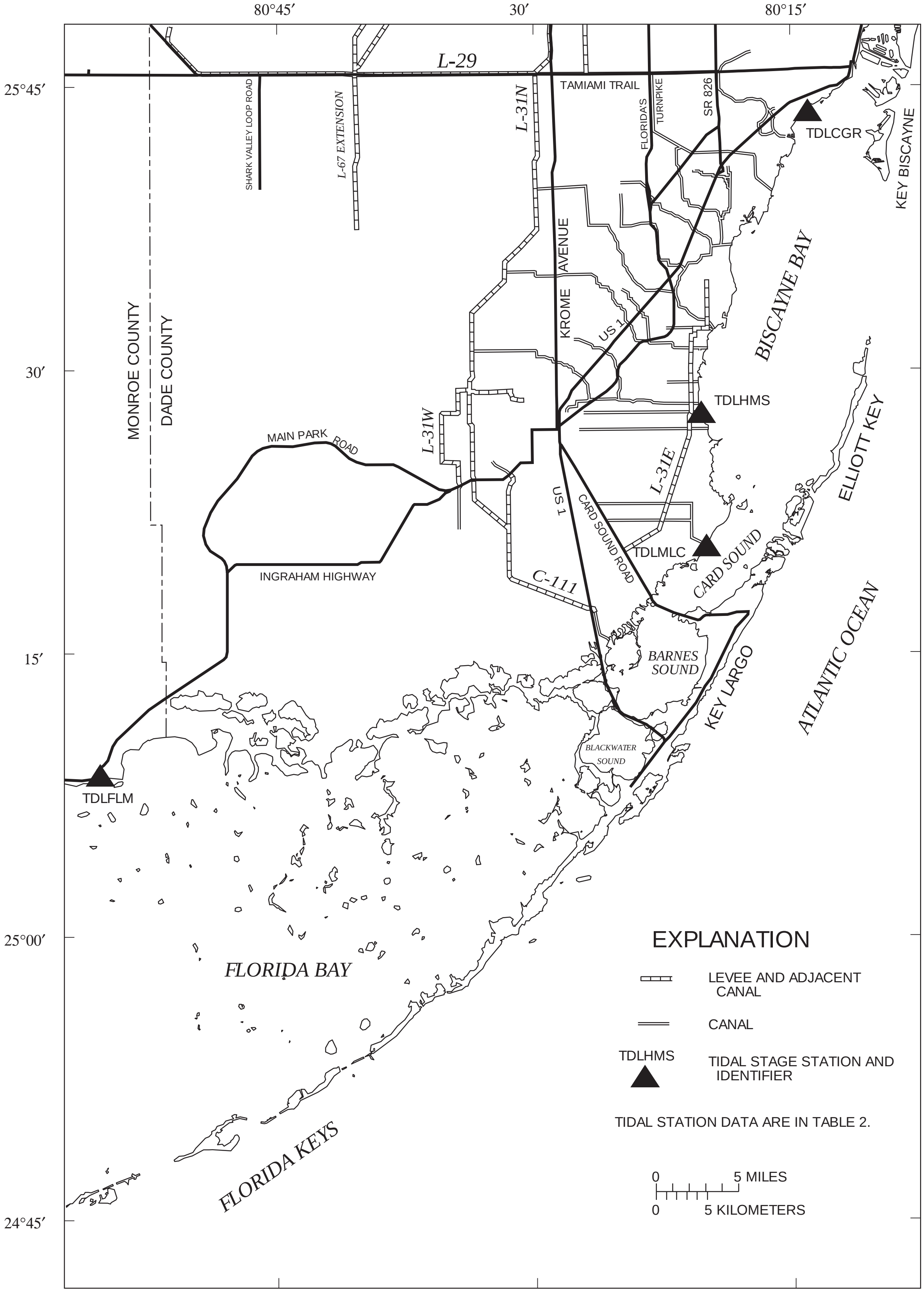


Figure 12. Locations of tidal gaging stations in southern Dade County used for model construction.

For an accurate comparison of tidal stages at the various sites, data from water years 1968 through 1980 were used to make 13-year averages for each month at each site. Two tidal cycles occur each day. The higher maximum is termed the “high,” and the lower is the “low high.” The lower minimum is the “low,” and the higher is the “high low.” Each monthly average was based on two- or four-sample daily averages, depending upon whether “daily high and low” values were stored (two-sample average), or whether “low high, low, high low, and high” values were stored (four-sample average).

At each tidal station, the average monthly stage (fig. 13) was found to have a 0.5 to 0.8 ft range of variation during the year, with the yearly low occurring between January and April and the yearly high occurring in October. Even more striking is the spatial trend of average tidal stages. During every month of the year, the average stage increases southward and westward long the coast. The stage is lowest at the most northern and eastern station at Coconut Grove (TDLCGR), higher at Bayfront Park near Homestead (TDLHMS), higher still in Card Sound (TDLMLC), and highest at Flamingo (TDLFLM). The month with the lowest average stage also varies from January to April between the sites at Coconut Grove and Flamingo.

The spatial trend might be related to the distance of the measuring site from the open ocean beyond the edge of the Atlantic Shelf on the eastern side of the barrier islands, which is least at Coconut Grove. Bayfront Park is more distant from the shelf (Uchupi, 1966) and is partly sheltered by Elliot Key. The Card Sound station is even more distant from the shelf and better sheltered. The Flamingo station connects to the Atlantic Ocean only through narrow channels between the Florida Keys, separated from the site by the broad, shallow expanse of Florida Bay, and is open to the Gulf of Mexico across an even broader expanse of shelf on the western side of the peninsula. The spatial trend might also be related to the configuration of the bottoms of the embayments.

No explanation of the seasonal variations is known to have been documented. The highest stages between September and December suggest a retarded response to wet-season (May-October) rainfall to the land area of Dade County, where the water table is highest between June and November. Discharges to the embayments by ground-water seepage or surface

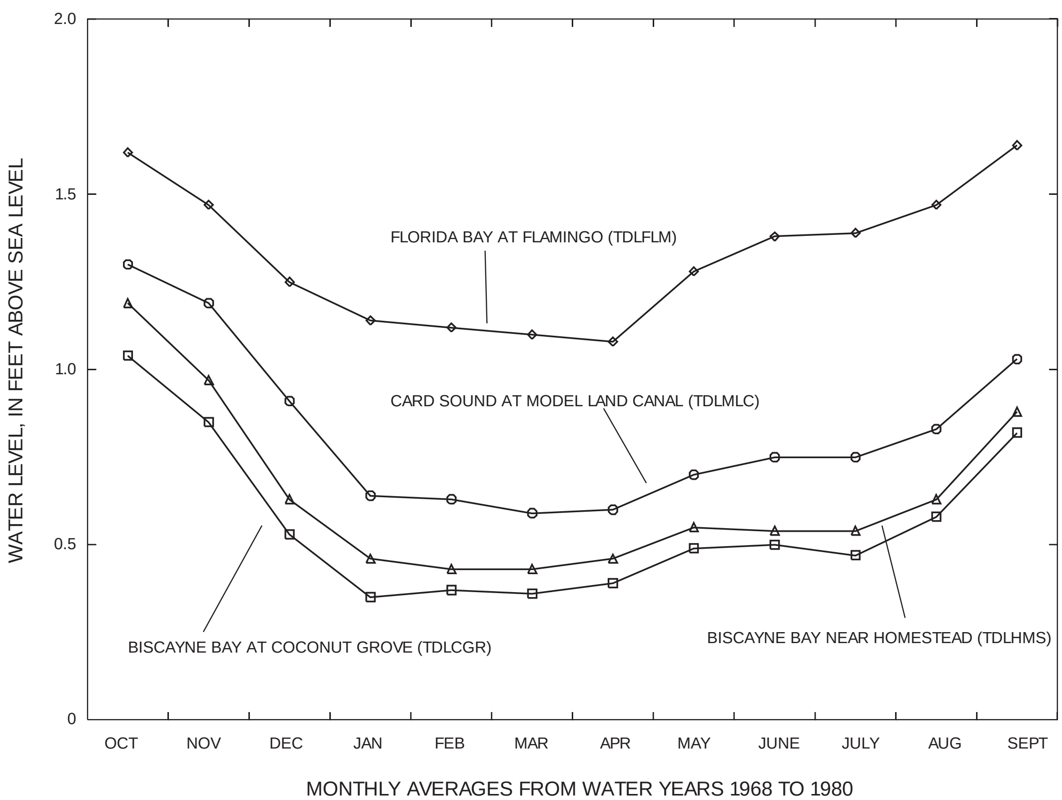


Figure 13. Long-term average monthly stages at tidal gaging stations in southern Dade County.

flows might cause average stages in these water bodies to peak slightly later. However, a detailed study of these hypotheses is beyond the scope of the present study.

Natural Ground-Water and Surface-Water Interactions

To design a ground-water flow model in the study area, it is necessary to assess the degree of interaction between the Biscayne aquifer and the natural surface-water flow system, consisting of seasonal shallow overland flows in the Everglades and southern glades (figs. 8 and 9). A standard technique for making a qualitative assessment of surface-water and ground-water interaction is by comparing changes in surface-water stages and water-table altitudes (referred to collectively as heads) when a stress is applied to one system or the other. In this study, the principal stresses in the nonurbanized parts of the study area, where surface flows occur, consist of vertical fluxes into or from the surface-water body (precipitation and evapotranspiration) and lateral fluxes in the surface- and ground-water bodies (the net residual of upstream and downstream drainage). If surface-water and ground-water bodies are hydraulically disconnected, a stress on one would not affect the head (surface-water stage or water-table altitude) of the other, or the effect would be significantly dampened or retarded, so that close correlation between the two heads as both change is evidence that the hydraulic influence of a stress on one is effectively transmitted to the other. Where local changes in surface-water stages and ground-water heads do not correlate well, the surface water and ground water are considered to be separated hydraulically or not to have a good hydraulic connection. One assumption of the present analysis is that wells are adequately grouted or otherwise sealed outside the casing from land surface to the bottom of the casing.

The most likely region for surface-water stages and ground-water heads to be hydraulically separated is in the peat- or marl-covered regions of Shark River Slough and the southern glades. Hydraulic separation is less probable in the rocky glades region where the aquifer extends to land surface. Therefore, it is in the glades region where surface-water and ground-water hydrographs were compared. Ideally, heads from closely adjacent stations should be used; however, only recently has such data been acquired by the USGS in the southern and eastern wetlands. A comparison in other parts of the region can only be accomplished using records from surface-water and ground-water stations located some distance from one another. This procedure has a drawback in that the intensity of rainfall events can vary appreciably over distance, and heads at specific times are not directly comparable because of their spatial variation. The procedure assumes that surface-water stage changes near the well are similar to those at the more distant surface-water stage measuring site.

The sites providing data for the comparisons are shown in figure 14. In upper Shark River Slough, data from the recorder in well G-620, cased to 6 ft and open to 16 ft, are compared with recorded stages at surface-water sites NP-203 and P-33. The time period for the comparison is October through December 1977, when water was above land surface at all three locations. Surface-water stages at NP-203 and P-33 and water levels in G-620 are shown in figure 15.

In well G-620, only a thin lens of permeable rock exists at about 6 to 10 ft below land surface; the well is west of the approximate western limit of the Biscayne aquifer as defined by Fish and Stewart (1991). Abrupt rises in water level show a close correlation with rainfall events of October 22, November 5 and 24, and December 6 and 17, as recorded by the rain gage at the Homestead Agricultural Experiment Station near well S-196A (fig. 3). Water-level responses below the peat layer in well G-620 closely parallel stage responses at surface-water sites NP-203 (2.8 mi away) and P-33 (5 mi away) and are of comparable magnitude. The evidence implies that the rock layer below 6 ft in G-620 is well connected hydraulically with flowing surface water in the vicinity.

Similar relations are observed at two other pairs of stations shown in figure 15, though the surface-water stage changes are dampened and more gradual compared to the ground-water level changes. Well NP-62 and gaging station NP-204 (fig. 14), 4.8 mi to the northeast of NP-62, are in the peaty region of central Shark River Slough. NP-62 is drilled to 20 ft and cased to 10 ft. Permeable rocks of the Biscayne aquifer extend from beneath the peat layer to about 25 ft below land surface at the site. NP-204 is beyond the western limit assigned to the Biscayne aquifer by Fish and Stewart (1991). Well NP-46 is about 4.1 mi southeast of gaging station P-38 (fig. 14). NP-46 and P-38 are both in the western part of the southern wetlands in the marly soil region, and heads at both sites are above land surface in the period shown.

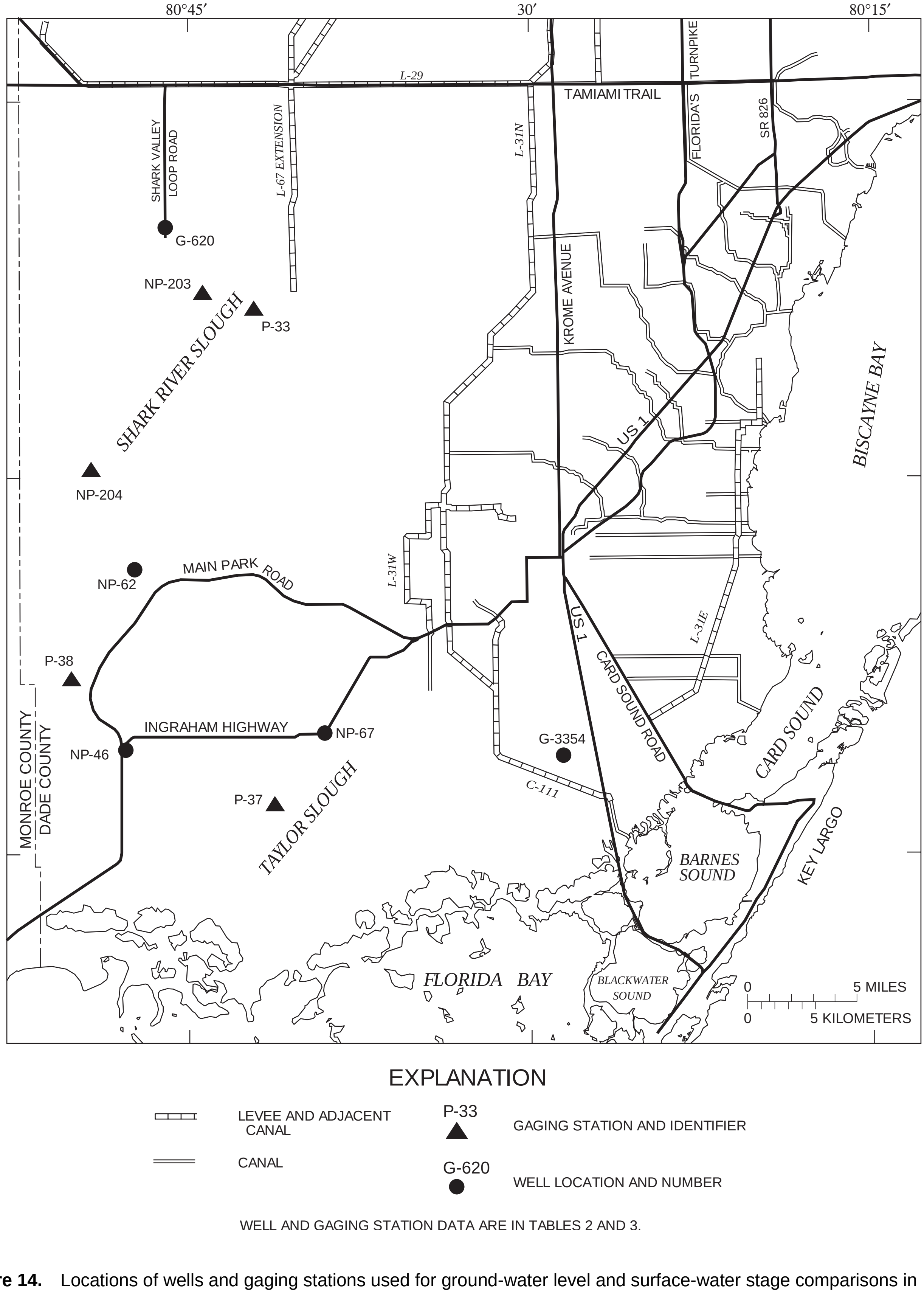


Figure 14. Locations of wells and gaging stations used for ground-water level and surface-water stage comparisons in peat- or marl-covered wetlands in southern Dade County.

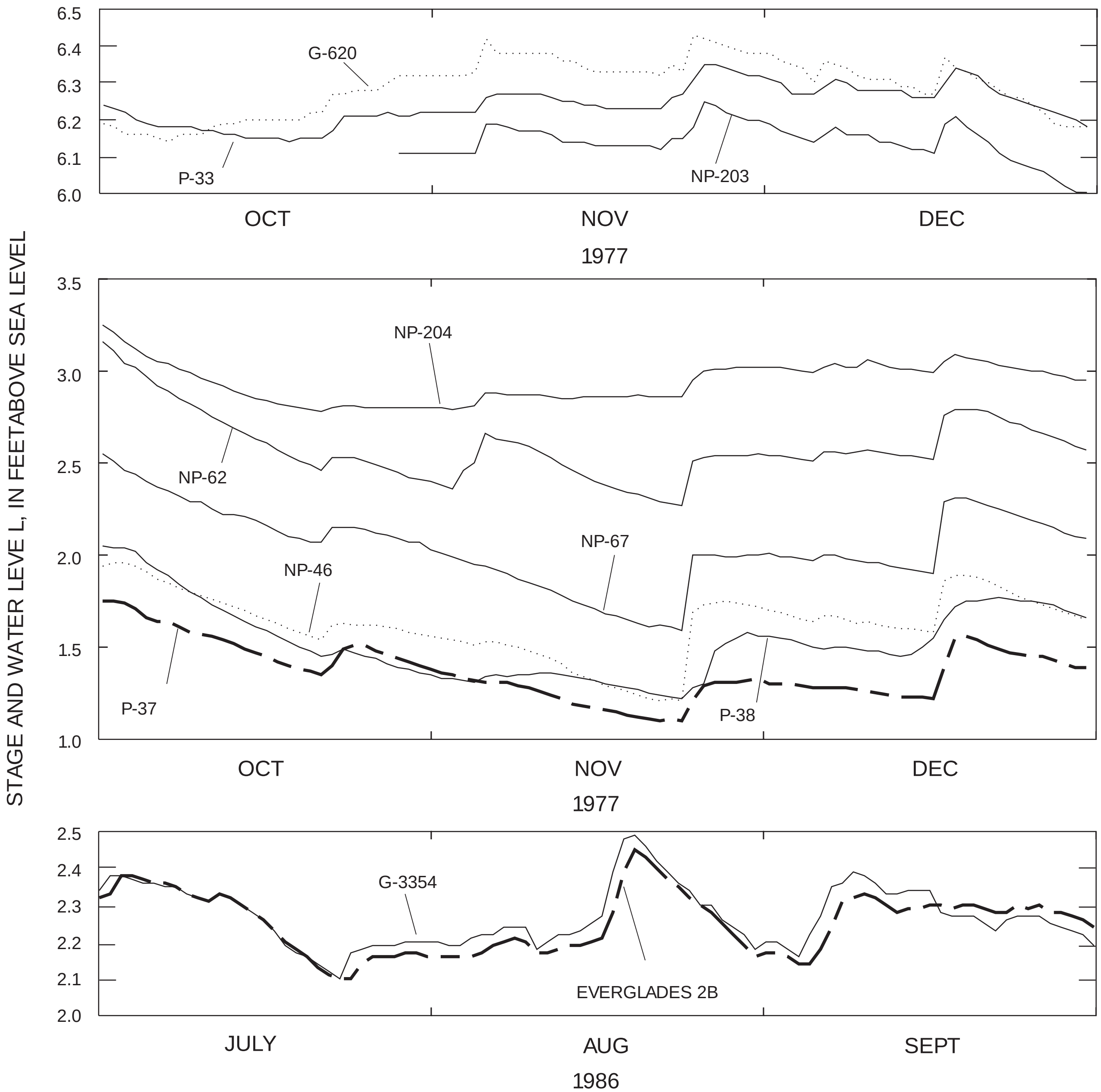


Figure 15. Ground-water levels and surface-water stages at selected pairs of wells and gaging stations in southern Dade County.

NP-46 is drilled to 25 ft and cased to 10 ft. Rocks of the Biscayne aquifer extend from beneath the marl to about 15 ft below land surface. The relative dampening of surface-water stage changes at NP-204 and P-38 may be related to their location farther toward the center of the slough and to the presence of more permeable rocks at the ground-water sites that transmit the effect of rapid water-table changes from nearby noninundated areas. Because of the distance between the surface-water and ground-water sites, confinement by the surface layer of peat or marl cannot be inferred from these data.

Well NP-67 and gaging station P-37 (fig. 14), about 3.8 mi to the south-southwest of NP-67, are both in the marly surface soil region of the southern glades slightly to the west of the main part of Taylor Slough. Heads in the time period shown (fig. 15) were above land surface at both locations, except possibly for a brief period in mid-November at NP-67. NP-67 is drilled to 20 ft and cased to 19 ft. The Biscayne aquifer extends from beneath the marl to about 50 ft below land surface. Head changes at NP-67 and P-37 are similar.

Paired surface-water stage recording station Everglades No. 2B and ground-water recorder well G-3354 were constructed in the marly soils region of the C-111 basin in 1985. Their location is indicated in figure 14 by the ground-water station number. Figure 15 shows heads from the two recorders in mid-1986 when the heads were above land surface. G-3354 is drilled to 8 ft and cased to 6.5 ft. The Biscayne aquifer extends from beneath the marl to about 60 ft below land surface. In the period shown, which is characteristic of the entire period of record, less than 1 in. separates the water levels. The slight differences could be related to the operation of C-111 control structures.

It seems that the Biscayne aquifer, or its thinning extension underneath northern Shark River Slough, has little or no confinement from surface water throughout Dade County south of the Tamiami Trail. The interchange of water probably occurs most readily in the rocky glades region where no peaty or marly layer is present. In the peat regions, circulation through the peat layer is slow, as was noted by Parker and others (1955, p. 109). However, when heads recorded daily or less frequently are compared, as described above, it is evident that no long-term confinement takes place. Stephens (1943) states a similar conclusion based on a comparison of heads from paired wells drilled in the Everglades by the Soil Conservation Service in 1942.

At a given location in the normally inundated regions with peat or marl surface deposits, ground-water recharge probably is only a small part of the local surface-water budget. This is because the resistance to surface-water flow is substantially less than resistance to downward (or upward) flow through the layer of peat or marl or to horizontal flow in the underlying aquifer. Recharge from flowing surface water and the consequent maintenance of ground-water levels virtually equivalent to surface-water stages, however, is of major significance to the water budget and flow regime of the Biscayne aquifer. Consideration of surface-water flow stages and related recharge of ground water should not be disregarded in attempts to design an accurate regional flow model of the Biscayne aquifer.

Water-Management System

The hydrologic regime of the Biscayne aquifer in southern Dade County has been substantially altered by man’s attempts to improve parts of the region for his own changing purposes by constructing a system of canals, levees, control structures, and pumping stations to manage surface-water and ground-water systems. The following sections briefly describe the major changes that have been implemented and their effects. The discussion is segmented into temporal periods corresponding to changes in water-management philosophy that have taken place for economic reasons or because of catastrophic events. The discussion of early events has a regional scope; the description of more recent events focuses on the study area, the southern two-thirds of Dade County.

Land Reclamation

In the mid-1840’s, following the end of armed conflict with Native American groups and the granting of statehood to Florida, proposals were made for reclamation of the vast areas of inundated lands in the southern part of the Florida Peninsula. The first proposals for arterial canals draining waters from the Everglades into the Gulf of Mexico or the Straits of Florida were advanced in the late 1840’s (Dovell, 1942). In 1850, the U.S. Congress transferred wetlands (“swamps and overflowed lands”) in various states to state control for the purpose of reclamation (Bestor, 1942). In 1855, the State of Florida created the Internal Improvement Fund whose trustees were empowered to sell wetlands for various purposes that would entail reclamation for economic benefit (Wallis, 1942).

In the next half century, title to most of the wetlands areas had been acquired by private interests, predominantly railroad companies, but little actual reclamation had occurred. In the early 1900’s, the Trustees of the Internal Improvement Fund reclaimed title to the wetlands in order that they might be disposed of for the express purpose of reclamation, and many years of litigation and financial uncertainty ensued. During this period, between the years 1905 and 1907, the dredging of the North New River Canal began. Laws passed in 1905 and 1907 created a Board of Drainage Commissioners to administer funds for reclamation activities within the boundaries of the newly established Everglades Drainage District. The Wright report of 1909 proposed the construction of regional drainage canals from Lake Okeechobee to the southeastern coast at locations extending south from West Palm Beach to Miami.

The resolution of financial and legal problems in 1913 accelerated the pace of the dredging of the regional drainage canals and other canals for local drainage in areas near the southeastern coast. By 1924, the regional drainage canals (fig. 1) were completed. During that year, the St. Lucie Canal was completed to the east, providing the first effective drainage of the lake, as the older West Palm Beach, Hillsboro, North New River, and Miami Canals were considered in-adequate to provide wet-season flood control (Parker and others, 1955, p. 334). Additionally, by 1924, a low rock and muck levee was constructed on the southern and eastern shores of Lake Okeechobee to prevent sea-sonal flooding and promote agriculture south of the lake. The levee was breached after hurricanes in 1926 and 1928, allowing flooding to occur that resulted in great destruction and loss of life (Parker and others, 1955, p. 321).

The hurricane of 1926 and the consequent collapse of the Florida land boom, followed by the hurricane of 1928 and the years of the Great Depres-sion, undermined the financial base of the Everglades Drainage District. It became insolvent in 1931 and funded no further construction until it was refinanced in 1941. Federal funds and the assistance of the U.S. Army Corps of Engineers were provided for the con-struction of a high levee around Lake Okeechobee and the improvement of the Caloosahatchee River and Canal to provide additional drainage capacity and stage control for Lake Okeechobee. Both projects were completed in 1937.

The construction of the Lake Okeechobee levee and the reduction of lake flood stages by the drainage canals and, to some extent, local drainage by the major canals and networks of secondary canals generally are considered to have had a substantial effect on the water-table altitude in the eastern and central parts of the peninsula south of the lake. The land area covered by wetlands has diminished, although data to accu-rately quantify changes are lacking. Parker and others (1955, p. 9) state that “construction of the drainage canals has lowered the average level several feet, not only in the Everglades but also in the coastal ridge.” The coastal ridge, though higher in elevation than the Everglades, lies to the east and hydraulically down-gradient. Southern Dade County (figs. 12 and 14) is distant from the drainage canals and the Lake Okeechobee levee, but the water table in southern Dade County might have been appreciably affected by the reduced stages and amounts of water flowing south through the Everglades flow system.

In 1916-17, a 60-mi long section of the Tamiami Trail, extending due west of Miami and transecting the area shown in figure 16, was completed using fill material from an adjacent borrow canal, excavated on the north side of the road (Douglas, 1947). An addi-tional section making an angle to the 1916 road west of Humble Road was completed in 1928. The system of culverts through which southward surface flows were allowed to pass under the roadway is shown in figure 10. The canal was considered generally non-arterial by J.H. Hartwell (U.S. Geological Survey, written commun., 1953) and to serve primarily as a collector and distributor of overland flows from the north (Parker and others, 1955, p 486-487). Stages could vary 1.5 ft between Forty-Mile Bend (fig. 16, TC40MB) and Bridge 45 (fig. 16, TCBR45) during high-water periods (D.B. Bogart, U.S. Geological Survey, written commun., 1946; Leach and others, 1972, p. 34). Sections of the canal were often clogged by weeds according to J.H. Hartwell (U.S. Geological Survey, written commun., 1953). From the early 1940’s and possibly earlier, the canal was dammed at the Dade-Broward Levee. A pair of corrugated metal pipes, probably installed in 1943, allowed discharge to occur at a stage of about 6 ft. According to Parker and others (1955), these pipes were replaced by solid fill in 1946. The dam was replaced with gated culverts in December 1954.

Just south of the Tamiami Trail, near the coast southwest of early urban Miami, the tidal Snapper Creek Canal (C-2) was constructed in 1912-13, and the Coral Gables Canal was constructed between 1925 and 1942 (Dade County, written commun., undated). A control was placed near the mouth of Snapper Creek Canal in 1946. The location of these canals is illus-trated in figure 16, which shows significant hydrologic features and principal traffic arteries during the main part of the 1945-52 period. A list of selected major and minor canals constructed in or near the study area until the present time (1993) and related data are presented in table 3.

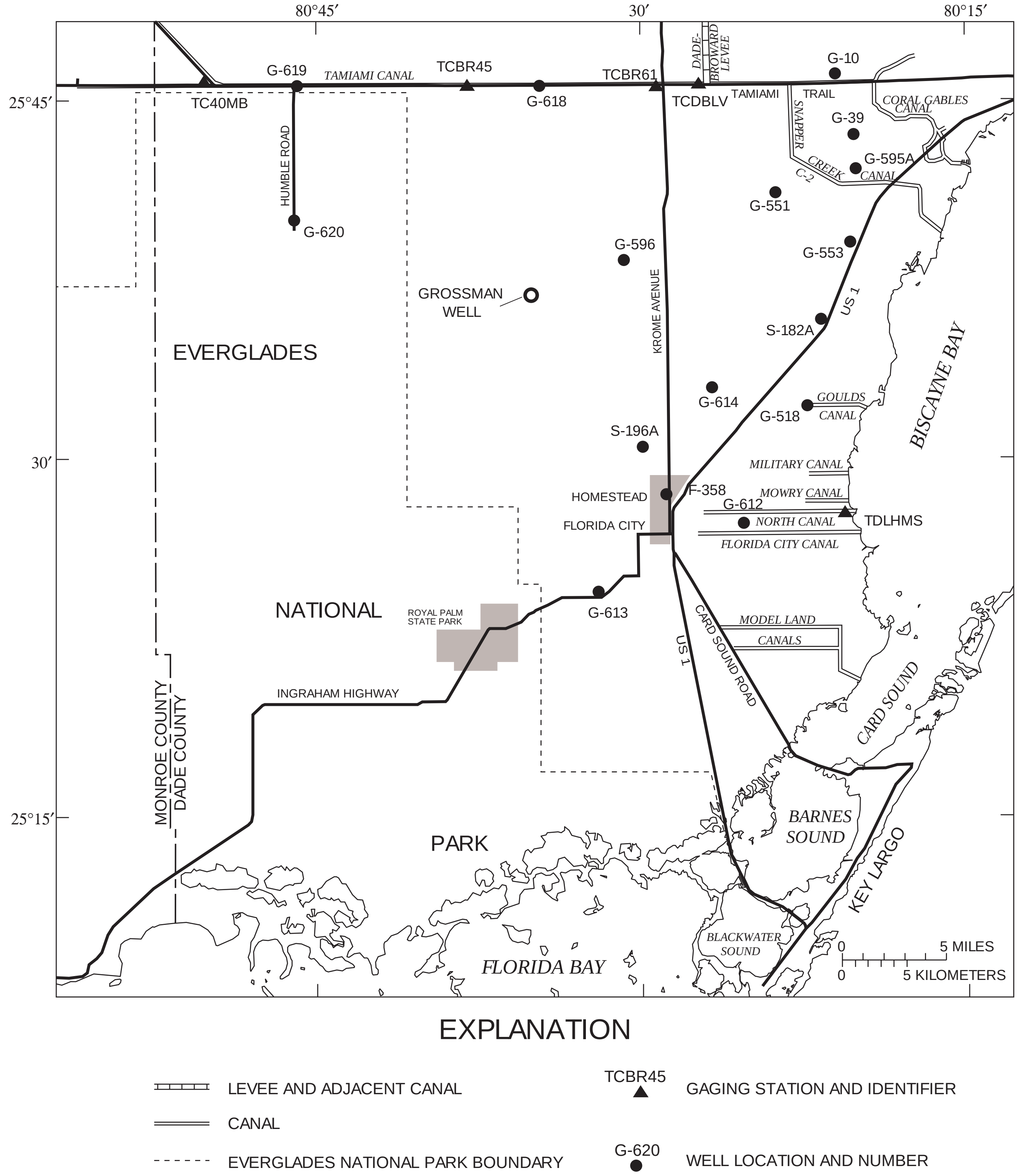


Figure 16. The canal-levee system in southern Dade County as it existed in water years 1945-52, and locations of wells and gaging stations providing continuous water-level and stage data during that period.

Table 3. Major drainage canals and levees in southern Dade County and selected secondary canals important to the flow model

[Construction information compiled from U.S. Geological Survey (USGS) literature and U.S. Army Corps of Engineers, Dade County, and South Florida Water Management District (SFWMD) records. Non-USGS sources do not guarantee accuracy of completion dates. —, not applicable or dates unknown]

SFWMD designation	Local name	Construction date	SFWMD[1] acceptance date	All plugs[2] removed	Purpose and remarks
C-2	Snapper Creek Canal	1912-13	—	—	Drainage
	Goulds Canal	Before 1921	—	—	Drainage agricultural lands
	Military Canal	Probably early 1940's	—	—	Drainage agricultural lands and Homestead Air Force Base
	North Canal	Before 1921	—	—	Drain agricultural lands; originally called Homestead Canal
	Florida City Canal	Completed 1912	—	—	Drain agricultural lands
	Campbell Canal	Probably late 1930's	—	—	Drain agricultural lands
C-107	Model Land Canals	Probably 1920's	—	—	Drain marshes for housing
C-4	Tamiami Canal: a. Miami to Pinecrest area b. Forty-Mile Bend to Naples	 1916-17 ?/27-04/28	 — —	 — —	Borrow canal for road construction. In 1962, structure S-12E in L-29 separated east and west sections of canal. East section deepened and widened with construction of L-29 in 1962 and again for South Dade Conveyance System in 1978. West section used for distributing flows to Everglades National Park.
C-3	Coral Gables Canal	1925-42	—	—	Drainage
	Dade-Broward Levee (and borrow canals)	1927	—	—	Flood protection
L-30	North-South Levee (and borrow canals)	05/52-09/52	09/52	—	Flood protection
L-31N	North-South Levee (and borrow canals)	05/52-09/52	10/52	—	Flood protection
	Grossman Road borrow canal	Probably 1961	—	—	Borrow canal for road construction
C-1 (C-1W)	Black Creek Canal	1961-62	—	—	Drain urban areas and prevent saltwater intrusion. Lower 5 miles channelized before 1921; called "Black Point Canal."
C-1N	Belaire Canal	1961-62	—	—	Drain urban areas and prevent saltwater intrusion
L-29	Levee and adjacent canal	Completed 12/62	—	—	Close Conservation Areas 3A and 3B. Constructed from east to west. Canal was linked through structure S-333 to eastern part of Tamiami Canal in 1978.
L-67A	Levee and borrow canal	1962-63	—	—	Stage reduction levee
L-67C	Levee and borrow canal	1962-63	—	—	Stage reduction levee
L-31E	Levee and adjacent canal: a. North of Goulds Canal b. South of Goulds Canal	 06/63-? 1965-05/67	 01/64 —	 — 08/67	Provide protection from storm tides. Constructed in stages from north to south (south of Goulds Canal). Section between Black Creek Canal and Goulds Canal (87th Avenue Canal) existed earlier.
C-100	Cutler Drain Canal	06/64-02/65	10/65	12/65	Drain urban areas and prevent saltwater intrusion
C-100A	Cutler Drain Canal	04/65-07/66	—	04/67	Drain urban areas and prevent saltwater intrusion
C-100B	Cutler Drain Canal	06/64-02/65	—	—	Drain urban areas and prevent saltwater intrusion
C-100C	Cutler Drain Canal	04/65-07/66	—	04/67	Same as above. Plug removed at upper end in March 1968.
C-102	Princeton Canal	05/65-04/66	09/66	06/67	Drain urban areas and prevent saltwater intrusion. Construction started at coast.
C-102N	Princeton Canal: a. Below S-195 b. Above S-195	 08/65-12/65 ?-10/70	 — —	 — —	Drain urban areas and prevent saltwater intrusion

Table 3. Major drainage canals and levees in southern Dade County and selected secondary canals important to the flow model —Continued

[Construction information compiled from U.S. Geological Survey (USGS) literature and U.S. Army Corps of Engineers, Dade County, and South Florida Water Management District (SFWMD) records. Non-USGS sources do not guarantee accuracy of completion dates. —, not applicable or dates unknown]

SFWMD designation	Local name	Construction date	SFWMD[1] acceptance date	All plugs[2] removed	Purpose and remarks
C-103	Mowry Canal	01/66-09/66	08/67	10/67	Drain urban and agricultural areas and prevent saltwater intrusion. Construction began 2 miles from coast in 1966; coastal section probably completed in late 1930's.
C-103S	Mowry Canal	04/66-06/66	08/67	10/67	Same as above
C-103N	Mowry Canal	05/66-03/67	08/67	10/67	Same as above
C-113	Levee and canal: a. Mile 1 b. Remainder	 05/66-07/66 07/68-12/68	 08/67 08/67	 — —	Drain agricultural lands; recharges Homestead, Florida City, and Florida Keys Aqueduct Authority Well Fields. Constructed west-east.
L-31 Remainder	L-31N (S-173 to S-176)	05/66-07/67	08/67	10/67	Flood protection; plug removed at S-173 in 12/68.
C-111[3]	Levee and canal: a. Section 3: S-176 to S-177 b. Section 2: S-177 to S-18C c. Section 1: S-18C to US 1* US 1 to Barnes Sound: a. Lower mile b. Upper mile**	 05/66-01/67 07/66-05/67 05/64-03/66 ?-09/66 ?-11/69	 08/67 08/67 04/66 — —	 05/67 06/67 — 01/69 —	Flood protection and removal of floodwaters
L-31W	Levee and canal	?-11/69	—	—	Water deliveries to Taylor Slough by way of pump station S-332, structure S-175, and cutaways in L-31W west of S-175.
L-28	Levee and canal	1963-07/67	—	—	Western closure of Conservation Area 3A
C-111E	Levee and canal	05/67-07/67	07/67	—	
C-110	Levee and canal	?-02/72	—	—	Never used. Still plugged. Construction of C-109 and C-110 was partly completed when saltwater intrusion into lower reaches caused plans for use for drainage to be changed (Meyer, 1974).
C-109	Levee and canal	?-02/72	—	—	Same as above
L-67 Extension	Levee and canal	06/66-11/67	—	—	Canal expedited delivery of Tamiami Canal water to Shark River Slough; levee confined surface flows to slough area. Canal culverts have been closed since June 1984.
L-29	Eastern section of L-29 borrow canal (Tamiami Canal) widening and deepening, L-67A to L-30	Completed 1977?	—	—	Water deliveries to southern Dade County
L-31N	Widening and deepening, Tamiami Trail to S-176: a. L-31N (to S-173) b. L-31N Remainder (S-173 to S-176)	 06/77-12/77 09/77-11/78	 — —	 — —	— Same as above Same as above

[1]When canals are constructed by non-SFWMD agencies, such as the U.S. Army Corps of Engineers for management by SFWMD, SFWMD formally "accepts" the canal after inspection to determine that construction specifications have been met.

[2]Canals are often constructed as a series of reaches separated by earthen plugs. When the canal is to be used by SFWMD for routing water, the plugs are removed.

[3]An asterisk (*) denotes gaps in south levee allow canal water to flow into the "panhandle" of Everglades National Park, and culverts in north levee allow surface flows into canal. A double asterisk (**) denotes postponed until S-197 constructed to prevent saltwater intrusion.

Farther south, extending to the coast from east of US 1 (fig. 16), are east-west canals once used to drain the low-lying coastal glades for production of winter crops and to provide for their transport to Biscayne Bay for shipment. According to Cross and others (1941), high-capacity, low-lift pumps in the canal dams were turned on near the end of the wet season in October, jetting water into Biscayne Bay to lower the water table quickly so that crops could be planted in subsequent weeks. The quantities of water discharged or pumped are unknown. The dams in the North and Florida City Canals (the others canals were uncontrolled according to Parker and others, 1955, p. 681) still existed near the ocean ends in 1995 though they were inoperable. The dams consisted of flapper gates that allowed free drainage when the canal stage exceeded the tidal stage, but closed when the tidal stage was higher than the canal stage to prevent saltwater influx into the canals that would cause saltwater to intrude into the aquifer. Despite the flapper gates, saltwater movement into the canals, bypassing the dams, during the drought of 1945 was documented by Parker and others (1955). Although not illustrated in figure 16, a vast network of secondary canals drained water from fields into the major canals. The Military Canal was probably constructed to drain the area surrounding Homestead Army Air Field before it was closed following major destruction caused by a hurricane in 1945.

Still farther south (fig. 16), the Model Land Canals were constructed in the 1920's by Henry Flagler's Model Land Company, which planned a residential development in the area of the canals that never achieved reality. Shallow borrow canals paralleled old US 1 (now Card Sound Road) and the newer extension of US 1 to Key Largo, completed in the late 1940's on the bed of Henry Flagler's railroad to Key West that was destroyed by a hurricane in 1935. A canal constructed in 1922 paralleled the Ingraham Highway from southwest of Florida City through Royal Palm State Park (fig. 16) to Cape Sable, on the southwestern part of mainland Monroe County (fig. 1), and some sections still exist. Called the Homestead Canal, it was intended to provide for barge traffic from Homestead to Cape Sable, but was apparently never completed through Homestead and Florida City.

Such were the water-management conditions prevailing when the flowing well was drilled at Grossman Hammock in late 1944. Figure 16 also shows 16 wells and 5 gaging stations that provided water-level or stage data for at least part of the 1945-52 time period. All are either near metropolitan Miami or are close to the major roads that existed during this period of time. Little or no data were obtained from the Everglades south of the Tamiami Trail. Site identification and other descriptive data are provided in table 2.

Flood Protection

In the early 1940's, scientific studies of the Everglades region were completed by the USGS, the Soil Conservation Service of the U.S. Department of Agriculture, and the University of Florida. The USGS studies (Parker, 1942; Ferguson, 1943; Parker and Hoy, 1943) defined the geologic framework and the basic features of the ground-water and surface-water systems of the area. Studies by the Soil Conservation Service (Stephens, 1943) defined the soil characteristics of the region and noted variations that had significance from the standpoint of reclamation. Whereas the Everglades region had been previously considered in its entirety as reclaimable for agricultural development, the Soil Conservation Service noted that only the northern part was suitable for agriculture because of the thickness of the peat soils and because the low permeability of the underlying rock made possible the effective local control of water. In the central and southern part of the region, the peat layer was thin and the high permeability of the underlying aquifer meant that water control would be feasible only on a regional scale. The Soil Conservation Service recommended that the central and southern part of the region be set aside as "water-conservation areas" and for the preservation of wildlife in Everglades National Park, authorized by the U.S. Congress in 1934. By 1947, three water-conservation areas had been established by the Everglades Drainage District. Everglades National Park was opened in 1947, and Royal Palm State Park was incorporated into the new Federal Park.

From 1943 to 1945, rainfall amounts were well below average in southeastern Florida. During this period, the poorly controlled drainage of Lake Okeechobee and of coastal lands to the southeast of the lake lowered the coastal water table to the extent that saltwater intruded into the aquifer, and major well fields faced the possibility of curtailment of pumpage. Emergency control structures were constructed in the lower reaches of the drainage canals near the ocean in 1946 and were replaced with permanent structures in subsequent years. The practice of uncontrolled drainage began to be reevaluated.

Hurricanes in 1946, 1947, and 1948 raised the water table to near land surface in urban areas, and in October 1947, water flowing eastward from the Everglades caused severe and long-lasting flooding in Dade and Broward Counties. It was then recognized that the drainage canals and the Dade-Broward Levee were insufficient to protect urban areas from flooding after intense and sustained rainfall. In June 1948, the U.S. Congress responded to the problem with the passage of the Flood Control Act that established the Central and Southern Florida Flood Control Project, a cooperative effort between the U.S. Army Corps of Engineers and local agencies. The Central and Southern Florida Flood Control District was created by the State of Florida in 1949 to manage and operate the proposed canal-levee system, and the emphasis in the conception of public works related to water shifted from land reclamation to flood control and water management (Leach and others, 1972).

The first phase was the construction of the north-south levee along the eastern edge of the Everglades. The levee crossed and plugged the Tamiami Canal about 1 mi west of Krome Avenue and extended south for about 10 mi where it ended in the somewhat higher elevation lands of the rocky glades. North of the Tamiami Trail, the levee was designated L-30, and south of the road, it was L-31N. A structure (S-24) at the Tamiami Trail permitted controlled releases from the L-30 borrow canal into the L-31N borrow canal. A seldom-used structure (S-24A), 3 mi south of the Tamiami Trail, could be operated to release water from the L-31N borrow canal westward into the Everglades during severe storms.

The flood control project also included plans to enclose the central Everglades water-conservation areas with an east-west levee north of the Tamiami Trail and another levee extending north from Forty-Mile Bend. Four spillways were planned in the levee north of the Tamiami Trail to release waters southward into Everglades National Park. Other plans were to surround the southern coastal ridge with levees. The north-south levee would be extended southward where an east-west levee would connect it across the southern glades with a coastal storm protection levee in the low-lying eastern glades.

A permanent control structure (S-22) was placed near the ocean end of Snapper Creek Canal (C-2) (discharge data are available from February 1959). Table 4 lists control structures constructed in the canal-levee system in the study area and related data. A map of the study area showing L-30, L-31N, and other water-control public works existing from 1953 to about 1961 and showing 25 wells and 10 gaging stations providing water-level and stage data during at least part of this period is shown as figure 17. The geographic distribution of data sites was improved relative to that of the 1945-52 time period, though there were still no sites in the southeastern part of the study area nor in the central rocky glades. New surface-water sites in the Everglades were accessible by airboat.

Levee construction planned as part of the Central and Southern Florida Flood Control Project continued along the Tamiami Trail until L-29 was completed in 1962 from L-30 in the east to the future southern end of L-28, 2.6 mi northwest of Forty-Mile Bend (fig. 18). L-67A and L-67C were constructed diagonally across the Everglades, north of the Tamiami Trail. The construction of L-29, L-67A, and L-67C achieved the partial closure of WCA-3, divided by L-67A and L-67C into subarea 3A to the northwest and subarea 3B to the southeast. Area 3B is bounded on the east by L-30 and on the south by L-29. Separation of the two subareas permitted the stage in 3B to be maintained lower than the stage in 3A, lessening seepage across L-30 and L-29, while making it possible for large water deliveries to be made to the western section of L-29, between L-67A and future L-28, which were destined for Everglades National Park.

As shown in figure 18 (water years 1962-77), the western section of the levee was bounded on the north side by the L-29 borrow canal that was linked to the Tamiami Canal south of the levee by the four gated spillways (S-12A to S-12D), which opened to release overland flows collected by the L-29 borrow canal and the L-67A borrow canal into the Tamiami Canal. The L-67A canal also channeled releases from Lake Okeechobee through the Miami Canal (north of area portrayed in fig. 19 and not shown) to the S-12 structures. From the Tamiami Canal, flows could pass southward through the culverts in the old road and enter Everglades National Park, destined eventually to reach Shark River Slough farther south.

Water deliveries to this section of Everglades National Park could be augmented in several ways. Big Cypress Swamp waters collected in the Tamiami Canal northwest of Forty-Mile Bend and, after 1967, in the L-28 borrow canal could be released to the Tamiami Canal through structure S-14, about 200 to 300 ft west of S-12A (fig. 18). This option, however, has never been implemented. Additionally, water collected in the Tamiami Canal/L-29 borrow canal section between L-30 and L-67A could be released to the Tamiami Canal between L-67A and Forty-Mile Bend, and thence to Everglades National Park,

Table 4. Control structures used for management of canal flows in southern Dade County

[Information compiled from U.S. Geological Survey (USGS) literature and U.S. Army Corps of Engineers, Dade County, and South Florida Water Management District records. Additional data provided by Cooper and Lane (1987). Non-USGS sources do not guarantee accuracy of construction completion dates. Structure type: GC, gated culvert; GS, gated spillway; D, dam with stop logs; PS, pumping station. Method of operation: M, manual; A, automatic; T, telemetry. An asterisk (*) indicates operated by Dade County; dashes (—) indicate not applicable or dates unknown]

SFWMD designation	Structue type	No. of gates or pumps	Method[1] of operation	SFWMD acceptance date	First operational log	Purpose	Current[2] operational schedule	Past operational schedules and other comments
*	GC	2,3	M	—	—	Restrict flow in Tamiami Canal at the Dade-Broward Levee.	—	In 12/54, gated culverts replaced dam. Old dam existed before 1940. Structure was moved 2 miles west in the 1960's or 1970's.
S-12A	GS	6	M	02/71	12/62	Control discharges from Conservation Area 3A to Everglades National Park.	—	Releases to park have been determined by U.S. Army Corps of Engineers on a real-time basis as determined by currently prevailing water budget policy and current water availability.
S-12B	GS	6	M	02/71	12/62	Same as above.	—	Same as above.
S-12C	GS	6	M	02/71	12/62	Same as above.	—	Same as above.
S-12D	GS	6	M	02/71	12/62	Same as above.	—	Same as above.
S-12E	GS	4	M	02/71	12/62	Control releases from Tamiami Canal east of L-67 into L-67 Extended canal and Tamiami Canal (when S-12F open).	—	Closed since 2/71, partly to prevent northward flows
S-12F	GC	4	M	06/71	03/69	Control flow in Tamiami Canal between S-12C and S-12D.	—	Known to be leaky. Is used to limit flows to L-67 Extended canal (Wagner and Rosendahl, 1982).
S-14	GC	2	M	01/63	08/64	Allow discharges from Big Cypress Swamp into Everglades National Park.	—	Releases are into Tamiami Canal west of L-67A. Closed since 1972.
S-18	GS		A	—	—	Completed 2/72 in C-109. Never wired for operation.	—	Never in use.
S-18C	GS	2	A	04/66	11/65	Control releases to section 1 of C-111. Since 1983, helps to drain agricultural areas.	2.6/2.3/2.0 YR	Before 7/69, varied between 2.0 and 2.4. Manual opening frequent in recent years.
S-20	GS	1	A	02/68	05/68	Control flows in L-31E canal at Model Land Canals.	1.9/1.7/1.5 WS 1.5/1.4/1.2 DS	WS: 2.2/2.0/1.8, 9/76-?; 2.0/1.9/1.6, ?-5/81; 2.4/2.1/1.8, 5/81-1/87. Rarely opens.
S-20A	GS	1	A	02/68	05/68	Control flows in L-31E canal north of Model Land Canals.	—	WS: 2.2/1.7/1.5, 6/69. Power off 2/71, not operating. Left in closed position.
S-20F	GS	3	A,T	04/67	04/67	Salinity-control structure for C-103.	1.7/1.5/1.3 WS 1.4/1.2/1.0 DS	WS: 2.0/1.8/1.6 before 6/69; changes often from 1.7-2.2 and back since 6/69. DS: 1.6/1.4/1.2 before 10/77. Schedule changes often.
S-20G	GS	1	A,T	07/66	06/66	Salinity-control structure for Military Canal.	1.8/1.6/1.4 WS 1.4/1.2/1.0 DS	WS has varied from 2.4/2.2/2.0, 2.2/2.0/1.8, and 2.0/1.8/1.6. DS was 1.9/1.5/0.65 in 10/80. Schedule changes often.
S-21	GS	3	T	12/61	12/61	Salinity-control structure for C-1	2.4/2.2/2.0 WS 1.8/1.6/1.4 DS	DS: 1.4-1.5/1.2/1.0, 4/66-12/68; 2.0/1.5-1.8/1.3-1.6, to ? WS: 2.4/1.5-1.8/1.5, 5/66-? Schedule often changes.

Table 4. Control structures used for management of canal flows in southern Dade County —Continued

[Information compiled from U.S. Geological Survey (USGS) literature and U.S. Army Corps of Engineers, Dade County, and South Florida Water Management District records. Additional data provided by Cooper and Lane (1987). Non-USGS sources do not guarantee accuracy of construction completion dates. Structure type: GC, gated culvert; GS, gated spillway; D, dam with stop logs; PS, pumping station. Method of operation: M, manual; A, automatic; T, telemetry. An asterisk (*) indicates operated by Dade County; dashes (—) indicate not applicable or dates unknown]

SFWMD designation	Structure type	No. of gates or pumps	Method[1] of operation	SFWMD acceptance date	First operational log	Purpose	Current[2] operational schedule	Past operational schedules and other comments
S-21A	GS	2	A,T	09/66	06/66	Salinity-control structure for C-102.	2.2/2.0/1.8 WS 1.4/1.2/1.0 DS	WS has varied periodically between 2.2/2.0/1.8 and 1.9/1.7/1.5.
S-22	GS	2	T	08/64	05/58	Salinity-control structure for C-2	3.5/2.9/2.5 WS ? DS	Accepted by Dade County in 6/56.
S-24	GC	1	M	10/52	08/64	Control flows from L-30 borrow canal and east Tamiami Canal to L-31N borrow canal. Circumvented by new construction in 1979.	—	
S-24A	GC	2	M	10/52	08/64	Allow emergency discharges from L-31N borrow canal westward into the Everglades; rarely used.	—	No authorized openings; unauthorized openings occurred 5/66, 9-10/81.
S-24B	GC	2	M	07/63	—	Control releases from L-30 borrow canal to Tamiami Canal east of L-30.	—	Replaced by S-335 in 1978.
S-118	GC	1	M	08/65	09/65	Stage-divide structure for C-100 west of US 1.	4.9/3.7/3.5 YR	Rarely opened; last time was 10/82.
S-119	GC	1	A	—	02/66	Stage-divide structure for C-100C west of US 1.	4.6/4.3/4.0 YR	From 2/66-6/82: 5.4/5.1/4.8 YR or 5.4/4.7/4.5 YR.
S-120	GC	1	M	09/66	08/67	Stage-divide structure for C-100A west of US 1.	6.0/5.0/4.0 YR	
S-121	GC	1	M	12/65	12/65	Control flows between C-100 and C-2 basins.	—	
S-122	GC	3	M	05/65	05/68	Control flows between C-100 and C-1 basins.	—	Has virtually never been used.
S-123	GS	2	T	01/66	09/65	Salinity-control structure for C-100.	3.5/3.0/2.5 WS 2.4/2.0/1.6/DS	Rarely opens.
S-148	GS	2	A	12/62	04/62	Stage-divide structure for C-1W west of US 1.	5.5/4.5/3.7 YR	
S-149	GC	2	M	12/65	07/63	Stage-divide structure for C-1N west of US 1.	6.2/5.5/4.8 YR	Last opening 4/83.
S-165	GS	1	A	09/66	07/66	Stage-divide structure for C-102 west.	5.9/5.5/5.1 YR	
S-166	GS	1	A	10/67	07/67	Stage-divide structure in C-103N west of US 1.	5.7/5.1/4.9 YR	
S-167	GS	1	A	06/67	07/67	Stage-divide structure for C-103N west of US 1.	5.9/5.5/5.1 YR	
S-173	GC	1	M	08/67	07/67	Control release from L-31N to L-31N Remainder	—	Generally closed at a stage of 3.5-5.0. Under agricultural agreement until 1/88. Since 1981, operates in conjunction with pump station S-331.

Table 4. Control structures used for management of canal flows in southern Dade County —Continued

[Information compiled from U.S. Geological Survey (USGS) literature and U.S. Army Corps of Engineers, Dade County, and South Florida Water Management District records. Additional data provided by Cooper and Lane (1987). Non-USGS sources do not guarantee accuracy of construction completion dates. Structure type: GC, gated culvert; GS, gated spillway; D, dam with stop logs; PS, pumping station. Method of operation: M, manual; A, automatic; T, telemetry. An asterisk (*) indicates operated by Dade County; dashes (—) indicate not applicable or dates unknown]

SFWMD designation	Structue type	No. of gates or pumps	Method[1] of operation	SFWMD acceptance date	First operational log	Purpose	Current[2] operational schedule	Past operational schedules and other comments
S-174	GS	1	A,T	—	08/70	Control releases from L-31N Remainder to L-31W canal for augmenting flows in Taylor Slough.	4.7/4.5/4.3 YR	5.7/5.5/5.1, to 6/82; 4.9/4.6/4.4, to 7/85. Agricultural agreement required frequent manual operation until 1/88.
S-175	GC	3	M	—	08/70	Control releases from L-31W canal to Taylor Slough.	4.5-5.0 WS 3.5 DS	Closed until 2/78, used often afterward. Generally closed at stage of 3.5, to 11/82; at 3.0 WS, 2.5 DS to 1988, now 3.5 again. Under agricultural agreement until 1/88.
S-176	GS	1	A,T	08/67	07/67	Control releases from L-31N Remainder to section 3 of C-111.	4.5/4.3/4.1 WS 5.0/4.8/4.6 DS	WS: 6.0, to 7/79; 5.7, to 6/82; 5.0, to 8/84. DS: 5.5, to 3/85. Under agricultural agreement until 1/88. Usually open after 1/84.
S-177	GS	1	A	08/67	06/67	Control releases from sections 3 to 2 of C-111. Since 1983, helps to drain local agricultural areas.	4.2/3.9/3.6 WS 3.5/3.3/3.0 DS	WS: 5.8, to 6/82; 5.0, to 7/85; 4.7, to ? Under agricultural agreement until 1/88. Usually open after 1/84.
S-178	GC	2	M	08/67	07/67	Control flows from Loveland Slough into C-111E.	—	Operated in storm events; usually closed around 2.5.
S-179	GS	2	A	07/67	04/67	Divide stage in C-103 and control releases to lower reach of C-103 in agricultural region.	3.9/3.5/3.1 WS 3.1/2.9/2.7 DS	DS: 2.7/2.4/2.1, to 11/71.
S-194	GC	2	M	09/66	07/66	Flow-divide structure. Control releases from L-31N canal to C-102.	—	Manual openings at: 5.5-5.0, to 4/82; 5.0-5.5, 5/82-11/82; 4.5-5.0, 11/82-4/84. Usually open thereafter. Two gates after 2/79.
S-195	GC	1	M	09/66	07/66	Stage-divide structure for C-102N west of US 1.	—	Opened for storm events and usually closed at about 3.0 until 5/82; 2.0 thereafter.
S-196	GC	1	M	08/67	07/67	Flow-divide structure; control releases from L-31N canal to C-103.	—	Manual openings: 5.5-6.0, to 4/79; 5.0-5.5, to 4/82; 4.5-5.0, to 1/84. Usually open thereafter.
S-197	G	3, 13[3]	M	07/69	06/69	Control releases from section 1 of C-111 to Card Sound and prevents saltwater intrusion. Removal of plug permits high volume releases in flood events.	—	Usually opened when stage more than 1.5, closed when less than 1.0. Plug removed five times: 8/18-28/81, 9/27-10/81, 6/3-5/82, 7/23-24/85, 8/15-23/88. Ten more gates added in 1990.
S-199	G		A	—	—	Completed 2/72 in C-110. Never wired for operation.	—	Never in use.
S-223	D		M	—	—	Force westerly outflows in northern 7-mile reach of L-67 Extended.	—	Circumvented by natural channel since construction.
S-331	PS	3	—	02/83	10/82	Pump water southward in L-31N canal for water deliveries to southern Dade County.	—	Siphons can operate without power when pumps off. Pumps usually operated for flood control until 1991 when G-211 became operational. Pumps rarely used afterward.

Table 4. Control structures used for management of canal flows in southern Dade County —Continued

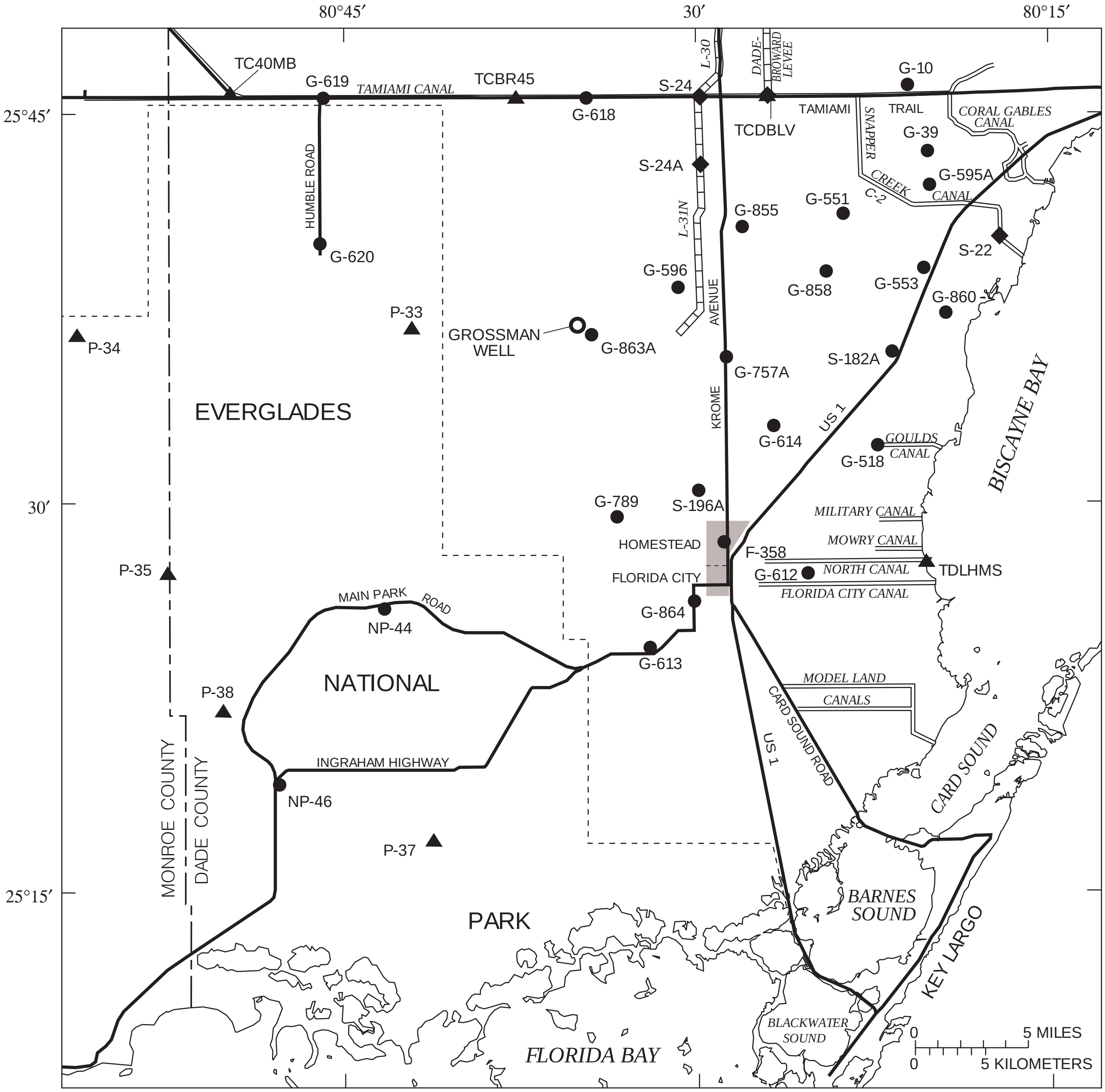
[Information compiled from U.S. Geological Survey (USGS) literature and U.S. Army Corps of Engineers, Dade County, and South Florida Water Management District records. Additional data provided by Cooper and Lane (1987). Non-USGS sources do not guarantee accuracy of construction completion dates. Structure type: GC, gated culvert; GS, gated spillway; D, dam with stop logs; PS, pumping station. Method of operation: M, manual; A, automatic; T, telemetry. An asterisk (*) indicates operated by Dade County; dashes (—) indicate not applicable or dates unknown]

SFWMD designation	Structuce type	No. of gates or pumps	Method[1] of operation	SFWMD acceptance date	First operational log	Purpose	Current[2] operational schedule	Past operational schedules and other comments
S-332	PS	6	—	08/80	01/81	Pump water from L-31W canal into Taylor Slough.	—	
S-333	GS		M	—	08/78	Control releases from L-29 borrow canal west of L-67A to L-29 borrow canal (Tamiami Canal) east of L-67A.	—	Part of South Dade Conveyance System. Conservation Area 3B waters and northern canal waters can be released to raise the water table in southern Dade County.
S-334	GS	1	—	—	04/78	Control releases from eastern section of L-29 borrow canal (Tamiami Canal) into L-31N for water deliveries to southern Dade County.	—	Part of South Dade Conveyance System. Releases generally made when stage more than 6.0 before 3/84, at stages of 4.5-5.5 thereafter.
S-335	GS	1	M	—	01/79	Control releases form L-30 into L-31N and L-29 borrow canals.	—	Generally closed at a stage of about 6.0 until 3/83, and at a stage of 5.0-5.5 thereafter.
S-336	GC	3	M	—	04/78	Control flows between Tamiami Canal east of L-30 (C-4) and L-30, L-31N, and L-29 borrow canals.	—	Difficult to generalize. Easterly/westerly releases can be made for flood control or flow augmentation purposes.
S-338	GC	2	M	—	03/79	Flow-divide structure. Control releases from L-31N into C-1W (Black Creek Canal).	—	Manual openings: 5.5-6.0, until 6/83; 5.0-6.0, until present.
S-346	GC	2	M	—	06/84	Control releases from Tamiami Canal at S-12D to L-67 Extended canal.	—	Normally closed. Since 1984, L-67 Extended has been blocked, and its original purpose abandoned. Current water policy is to redirect flows into L-29 borrow canal east of L-67A for release into Northeast Shark River Slough.
S-347	GC	2	M	—	06/84	Control flows in south end of L-67 Extended canal. Force water westward to Shark River Slough; otherwise, flows reaching south end would tend to flow eastward to Northeast Shark River Slough.	—	Normally closed. above comments apply.
G-211	GC	2	M	01/91	01/91	Restricts area drained by pumping S-331 in wet season months.	—	

[1]Telemetry after 10/77 for S-18C, S-20G, S-21A, and S-20F; after 1/88 for S-176 and S-174.

[2]Given as O/S/C, where O is setting at which structure opens, structure remains static at stage S and closes when stage drops to setting C. WS is wet setting (April 30, May 15, or May 30 to October 15); DS is dry setting (October 15 to April 30, May 15, or May 30); and YR is same setting all year round. Upstream stages in feet above sea level. October 15 is beginning of agricultural season.

[3]Thirteen gates after 1990.



EXPLANATION

- LEVEE AND ADJACENT CANAL
- CANAL
- EVERGLADES NATIONAL PARK BOUNDARY
- ▲ P-33 GAGING STATION AND IDENTIFIER
- ● G-620 WELL LOCATION AND NUMBER
- ◆ S-24 CONTROL STRUCTURE AND NUMBER

Figure 17. The canal-levee system in southern Dade County as it existed in water years 1953-61, and locations of wells and gaging stations providing continuous water-level and stage data during that period.

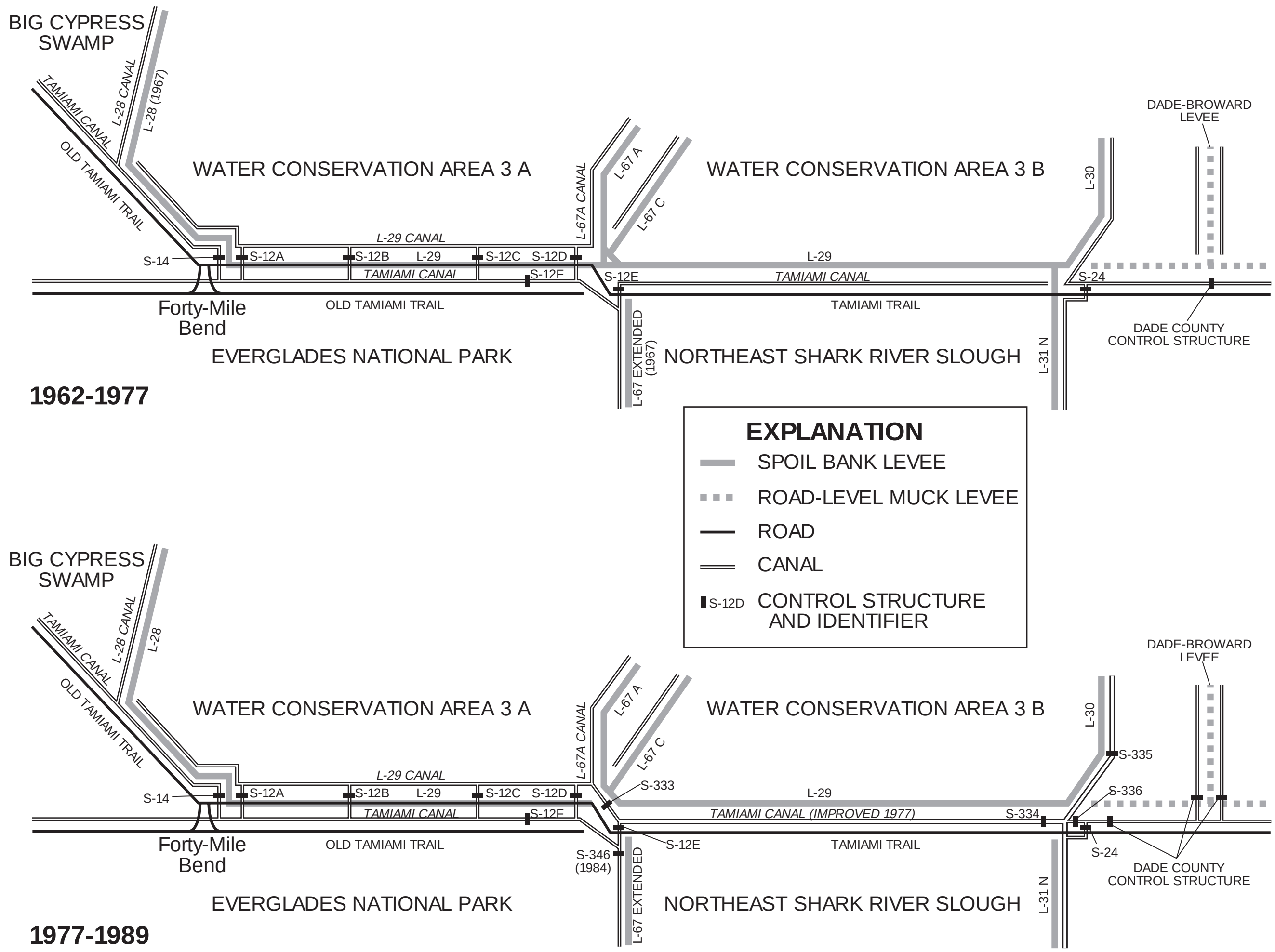
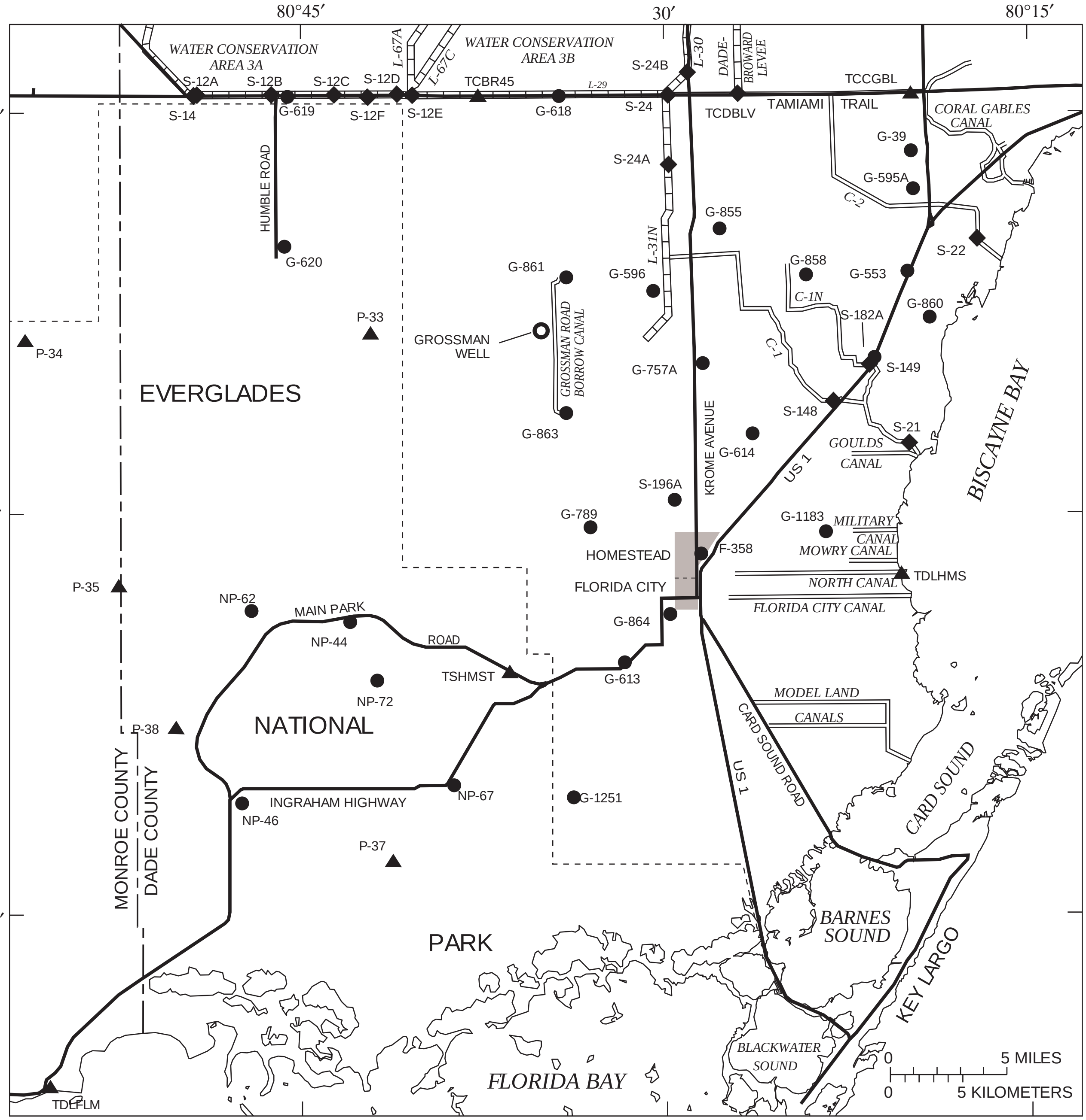


Figure 18. The system of surface-water control in the vicinity of the Tamiami Trail between Levee 28 and the Dade-Broward Levee, 1962-89 water years.

through structure S-12E. There were no spillways in the eastern section of L-29 between L-30 and L-67A. Thus, southward flow of surface water from WCA-3B into the Tamiami Canal between L-30 and L-67A could only occur by seepage under the levee. (The term “seepage levee” refers to such a levee that impounds surface water without releasing part of it through structures.) The seepage water could then pass southward as surface flow through culverts in the road.

With the later completion in July 1967 of sections of L-28 on the western side of WCA-3A, the system of levees allowed complete control (except for evaporation, percolation, and levee seepage) of surface waters in the central Everglades. Releases to Everglades National Park in the southern part of the Everglades were also largely controlled.

In 1961 and 1962, Black Creek Canal (C-1) and Belaire Canal (C-1N) were excavated in the northeastern part of the study area, where flooding had occurred during previous periods of intense rainfall. C-1 extended from the L-31N borrow canal to Biscayne Bay just north of the Goulds Canals, but was plugged off from the L-31N borrow canal. The designations C-1 and C-1W are synonymous north and west of the junction of C-1 (C-1W) and C-1N, which extended northward from the natural stream section of C-1. A coastal control structure (S-21) was built to control saltwater intrusion (table 4), and secondary controls (S-148 in C-1W and S-149 in C-1N) were constructed on the northwest side of US 1 to permit higher stages to be maintained upstream (stage divide structures). Control structures S-21 and S-148 were equipped with electrically operated automatic controls that were set to open or close the gates when the upstream stage rose above or below designated control elevations.



EXPLANATION

LEVEE AND ADJACENT CANAL		▲ P-33	GAGING STATION AND IDENTIFIER
CANAL		● G-620	WELL LOCATION AND NUMBER
EVERGLADES NATIONAL PARK BOUNDARY		◆ S-24	CONTROL STRUCTURE AND NUMBER

Figure 19. The canal-levee system in southern Dade County as it existed in water years 1962-67, and locations of wells and gaging stations providing continuous water-level and stage data during that period.

The state of the canal-levee system in the study area for several years after 1962 and well and gaging station coverages are shown in figure 19. The number of wells providing water-level data used in this study increased to 27; the number of gaging stations providing stage data increased to 13. The spatial distribution of data sites generally is excellent, except for the extreme southern and southeastern glades region and the rocky glades region north of the main park road in Everglades National Park. Also shown on the map is the borrow canal that paralleled Grossman Road on the approach to Grossman Hammock. The road was extended north and south to provide access to Federal Aviation Agency transmitting towers, probably in 1961. Narrow, shallow, and weed-choked, the canal has never been any different from thousands of other borrow ditches in Dade County. However, its proximity to the Grossman well, source of the high-chloride plume, required representation of the borrow canal in the model of the flow system.

Few, if any, releases were made through the S-12 structures from 1962 to 1964 because of prevailing drought conditions. In later years, experience gained in using the structures for releases to the park showed that southward flows near S-12A and S-12B were relatively small compared with southward flows in the vicinity of S-12C and S-12D because of higher land elevations and flatter land-surface gradient to the south near S-12A and S-12B (Wagner and Rosendahl, 1982). Some water released at S-12A and S-12B moved overland to the southwest rather than southward toward Shark River Slough.

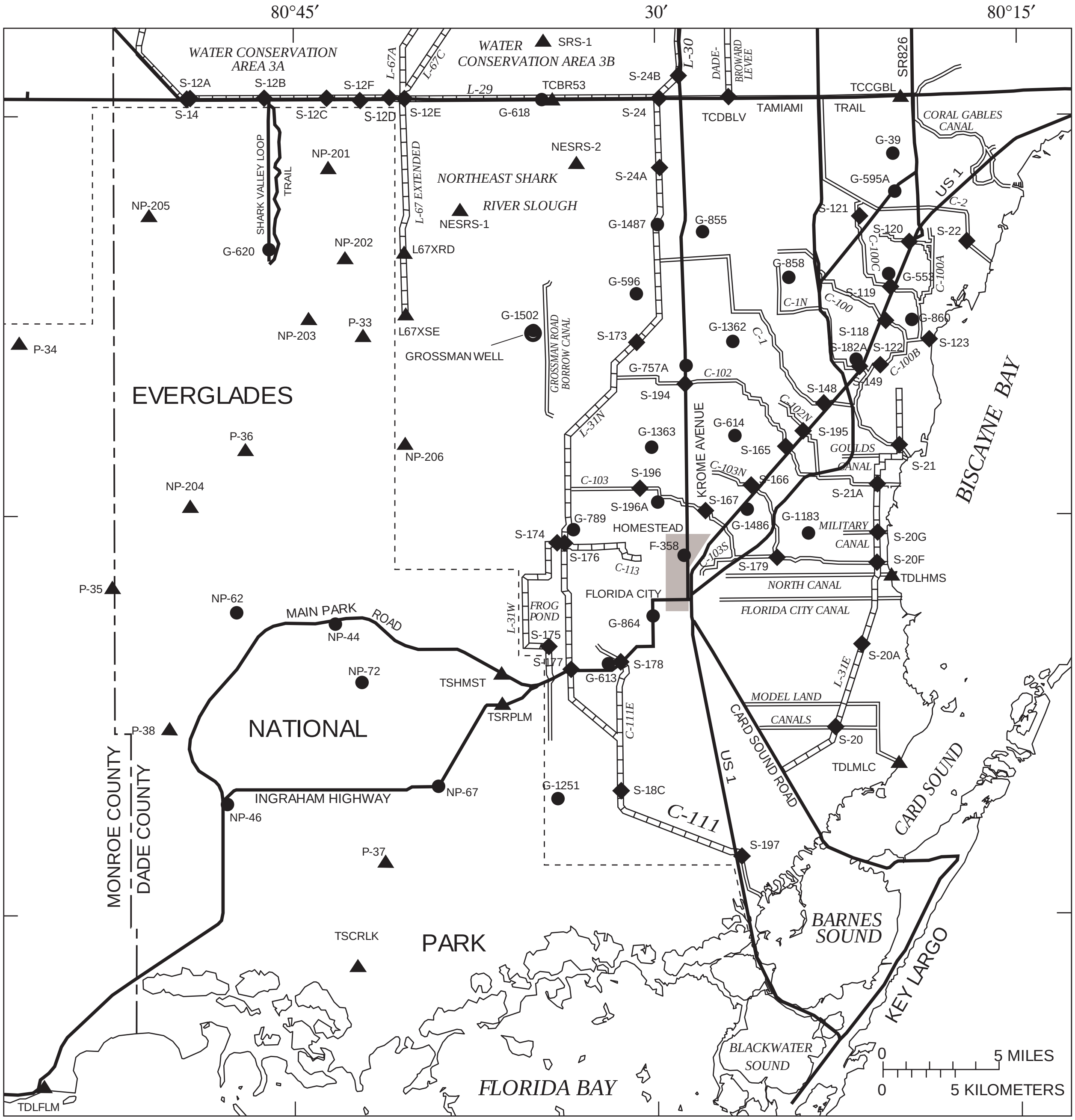
L-67 Extended was constructed in 1967 as an extension of L-67A south of the Tamiami Trail into the center of Shark River Slough. The borrow canal, west of the levee, was connected directly to the Tamiami Canal (fig. 18). The purpose of the canal was to facilitate water deliveries to Shark River Slough during drought periods (Neidrauer and Cooper, 1989, p. 4), and the purpose of the levee was to confine water deliveries from the S-12 structures to the area within the park boundaries.

Effects of the construction of L-67 Extended and its borrow canal have not been entirely beneficial. The levee lowers overland flow stages on its eastern side. As a result, water reaching the southern end of the canal sometimes moves northeastward into Northeast Shark River Slough, the area south of the Tamiami Trail between L-67 Extended and L-31N (Neidrauer and Cooper, 1989, p. 61). Additionally, the canal tends to drain sheetflow from the region of Everglades National Park south of the Tamiami Canal (Wagner and Rosendahl, 1982, p. 30). The canal also drains too much water directly from the Tamiami Canal, decreasing the amount that might otherwise move southward as sheetflow. Park ecosystems have, at times, been disturbed by the sudden deliveries of large quantities of water (Neidrauer and Cooper, 1989, p. 4; U.S. Army Corps of Engineers, 1992, p. 12), and the beneficial influence of slow-moving sheetflow on the quality of Tamiami Canal waters entering the slough is diminished. Structure S-12F in the Tamiami Canal between the S-12C and S-12D outlets has been used to reduce the amount of flow reaching the eastern extremity of that canal where it entered the uncontrolled L-67 Extended canal before 1964. A dam (S-223) was placed in the L-67 Extended canal, 7 mi south of Tamiami Trail, to force water out into the slough. However, moving waters scoured a channel that circumvented the structure (Wagner and Rosendahl, 1982, p. 30).

Flood Control in Southern Dade County

A series of tropical storms in 1960 that included Hurricane Donna caused severe flooding in southern Dade County, despite the presence of the north-south levee and control of waters in the central Everglades. In 1962, a dry period of unusually long duration caused the water table in southern Dade County to be lowered to an undesirable degree, despite high stages that were maintained in Black Creek Canal (C-1). The first problem, to provide flood protection for southern Dade County, was solved by the completion of the construction of a complex system of levees, canals, and control structures (fig. 20 and tables 3 and 4), by late 1967, at which time most of the temporary plugs were removed to make the system operational. The high waters of May and June 1968 were more effectively controlled than those of 1960 (Leach and others, 1972, p. 99).

As shown in figure 20, a southward extension of L-31N (L-31N Remainder) was constructed to form, with construction along C-111, a levee system extending continuously, except for several small breaches, from L-30 to within 2 mi of the coast at Barnes Sound. A coastal levee, L-31E, extending from the old US 1 (Card Sound Road) borrow canal to north of the mouth of Black Creek Canal (C-1), was completed in May 1967. This levee protects low-lying agricultural and residential land from flooding by storm tides.



EXPLANATION

LEVEE AND ADJACENT CANAL	▲P-33 GAGING STATION AND IDENTIFIER
CANAL	● G-620 WELL LOCATION AND NUMBER
EVERGLADES NATIONAL PARK BOUNDARY	◆S-24 CONTROL STRUCTURE AND NUMBER

Figure 20. The canal-levee system in southern Dade County as it existed in water years 1968-82, and locations of wells and gaging stations providing continuous water-level and stage data during that period.

The L-31E borrow canal connects the network of primary and secondary canals to the principal coastal salinity-control structures and serves as a distributed saltwater intrusion barrier.

Coastal lands north of L-31E are part of the higher elevation coastal ridge and generally are not subject to flooding. These rapidly urbanizing sections of land were drained after 1967 by the Cutler Drain Canal system (C-100, C-100A, C-100B, and C-100C). Coastal salinity-control structure S-123 (table 4) is closed to prevent overdrainage and saltwater intrusion during periods when the water table is low. Stage divide structures (S-119 and S-120) were constructed near US 1, and other controls (S-121 and S-122) manage interbasin drainage.

Between the two levees and south of Black Creek Canal (C-1), low-lying agricultural and residential lands east of the coastal ridge are drained by the Princeton Canal system (C-102 and C-102N) and the expanded Mowry Canal system (C-103, C-103N, and C-103S). C-103S does not extend as far west as originally planned, in response to the requests of local government officials in Homestead and Florida City. These canal systems protect adjacent lands from saltwater intrusion during dry periods by the operation of salinity-control structures in L-31E (S-20, S-20A, S-20F, S-20G, and S-21A). Stage-divide structures (S-165, S-195, S-166, and S-167) were constructed near US 1 to maintain higher stages upstream. An additional stage-divide structure (S-179) was constructed east of Homestead (fig. 20) where C-103 passed through agricultural lands. Wet- and dry-season stage criteria (wet and dry settings) were developed for the operation of new automatically controlled gates in the agricultural areas and near the coast.

Flow-divide structures (S-194 and S-196) were constructed about 3 mi east of junctions of C-102 and C-103 with the L-31N borrow canal to prevent eastward drainage of L-31N canal waters, except when it became desirable to release floodwaters into C-102 and C-103. Generally, this was permitted to occur only after the high water table resulting from storms had subsided in the residential and agricultural lands to the east so that releases to the canals would not cause additional flooding. Before 1983, these manually operated flow-divide structures were opened only 12 times.

The principal discharge points at or near the coast were at control structures S-22 in Snapper Creek Canal (C-2), S-123 at the mouth of Cutler Drain Canal (C-100), S-21 where Black Creek Canal (C-1) intersects L-31E, S-21A where Princeton Canal (C-102) intersects L-31E, S-20G where Military canal meets L-31E, and S-20F where Mowry Canal (C-103) intersects L-31E. Table 5 lists coastal discharge data sites and indicates the period for which discharge has been computed by either the USGS or the South Florida Water Management District.

Various structures (S-173, S-176, S-177, and S-18C) were emplaced in the L-31N canal and in C-111 to control stages in the various reaches of this long, continuous canal, extending from the Tamiami Canal to the southern coast. In the lower part of C-111 below S-18C, water was allowed to pass into the canal through culverts in the northeast levee and to discharge southwestward through cutaways in the southwest levee. The C-111 canal was originally intended to be uncontrolled south of S-18C. Major north-south canals C-109 and C-110 (not shown in fig. 20) between C-111 and US 1 were nearly completed, C-108 was planned for construction between US 1 and Card Sound Road, and improvements to the Model

Table 5. Stations providing discharge data used for flow-model calibration

[U.S. Geological Survey data are stored in computer files of the Miami Subdistrict Office. Period of record: 1990+ indicates that data collection was continuing at the time of the first draft of this report (1990). Agency: USGS, U.S. Geological Survey; SFWMD, South Florida Water Management District]

USGS identification	Location	Period of record	Agency	Use in model
02290700	Snapper Creek Canal at S-22 near south Miami	02/59-09/85	USGS	Boundary
		01/84-1990+	SFWMD	Boundary
02290725	Mowry Canal near Homestead (at S-20F)	03/69-1990+	USGS	Boundary
02290710	Black Creek Canal at S-21 near Goulds	10/69-1990+	USGS	Boundary
	S-123 (in Cutler Drain Canal, C-100)	12/79-1990+	SFWMD	Other
	S-331 (pumping station in L-31N)	11/82-1990+	SFWMD	Routing
	S-21A (in Princeton Canal, C-102)	12/82-1990+	SFWMD	Other
	S-20G (in Military Canal)	12/84-1990+	SFWMD	Other

Land Canals east of Card Sound Road were planned when Klein (1965) documented evidence that the planned canal system would cause saltwater encroachment much farther inland than that already occurring. In 1969, structure S-197 was placed in C-111 near US 1, 2 mi upstream of Barnes Sound, to control saltwater intrusion. Remaining canal construction planned for the southeastern glades was never completed, and C-109 and C-110 were plugged at their confluence with the lower reach of C-111.

The construction of L-31W and its borrow canal was completed in 1969. A control structure (S-174) was located in the new borrow canal 300 ft west of its origin at the junction of L-31N Remainder and C-111 and 300 ft north of structure S-176 in C-111. A second control structure (S-175) was located near the southern end of the borrow canal. Near the west end of the southern east-west section of the canal and upstream of S-175, two culverts allowed water to pass under the levee south of the canal. Stages in the canal were controlled to facilitate releases southward through the culverts to Taylor Slough (fig. 3), which was considered not to have received inflows equivalent to natural historic flows since 1962 because of the lowering of stages in the region of its source waters in Northeast Shark River Slough.

A six-unit pumping station (S-332) upstream of the cutaways (fig. 21) was completed in late 1980. After operation of the pumping station began the National Park Service established a requirement for 37,000 acre-ft/yr of water to be delivered to S-332 and to S-18C for release through the C-111 levee cutaways farther south. The culverts in L-31W west of S-175 have probably been largely ineffective because the local land elevation is high relative to that within the slough at S-332 and are now kept closed with stoplogs (D. Sikkema, National Park Service, oral commun., 1990).

Canal C-113 extended from its origin 275 ft south of structure S-176 in C-111 to the vicinity of Homestead and Florida City. During dry periods, waters released through S-176 while S-177 remained closed could replenish the water table near well fields used by these cities and by the Florida Keys Aqueduct Authority.

Figure 20 shows the location of 27 wells and 26 gaging stations operated during at least part of the 1968-82 time period and which provided data for model calibration. A record of the periods of operation of the flood-control system for drainage of floodwaters is shown in table 6. Typically, during flood events, manually operated structures that were normally closed were opened. Also, automatically controlled structures were electrically disconnected and operated manually. On several occasions, the system has been put on manual operation in advance of anticipated storms. This permitted early lowering of stages below control elevations so that the capacity of the system to remove floodwaters was enhanced when the flood stages occurred.

The flood-control system was operated manually twice in 1968 and twice more in 1969; then, nearly 10 years elapsed without severe flooding, until the freak April storm of 1979 once again required the manual operation of the system. From April 1979 through November 1982, the system was operated manually 10 times for flood control. Also, the high waters of late 1981 and late spring 1982 required the removal of the earthen plug in C-111 near S-197 on three occasions. This procedure was normally only undertaken during exceptionally severe flooding when the discharge capacity of the flood-control system needed to be augmented.

It should be noted that table 6 represents the generalization of a large data base. Individual structures might have been operated manually for longer or shorter periods than shown in the table, particularly when the manual operation period was lengthy.

Water Management in Southern and Eastern Dade County

Construction began in the late 1970’s for implementation of a water-management plan to: (1) mitigate the effects of prolonged dry periods on the water-table elevation in southern Dade County; and (2) further facilitate the removal of floodwaters of the scale of those caused by Hurricane Donna in September 1960. Planners envisaged the delivery of additional quantities of water to augment flows to the southeastern part of Everglades National Park (Neidrauer and Cooper, 1989, p. 4) and to prevent low water levels from occurring near regional well fields.

The water-management plan did not require the construction of any new canals, but did require the improvement of some existing canals and the operation of new control structures to move large quantities of water from WCA-3A into southern Dade County. The shallow Tamiami Canal between L-30 and L-67A was widened and deepened, as was the L-31N canal from Tamiami Trail south to its juncture with C-111.

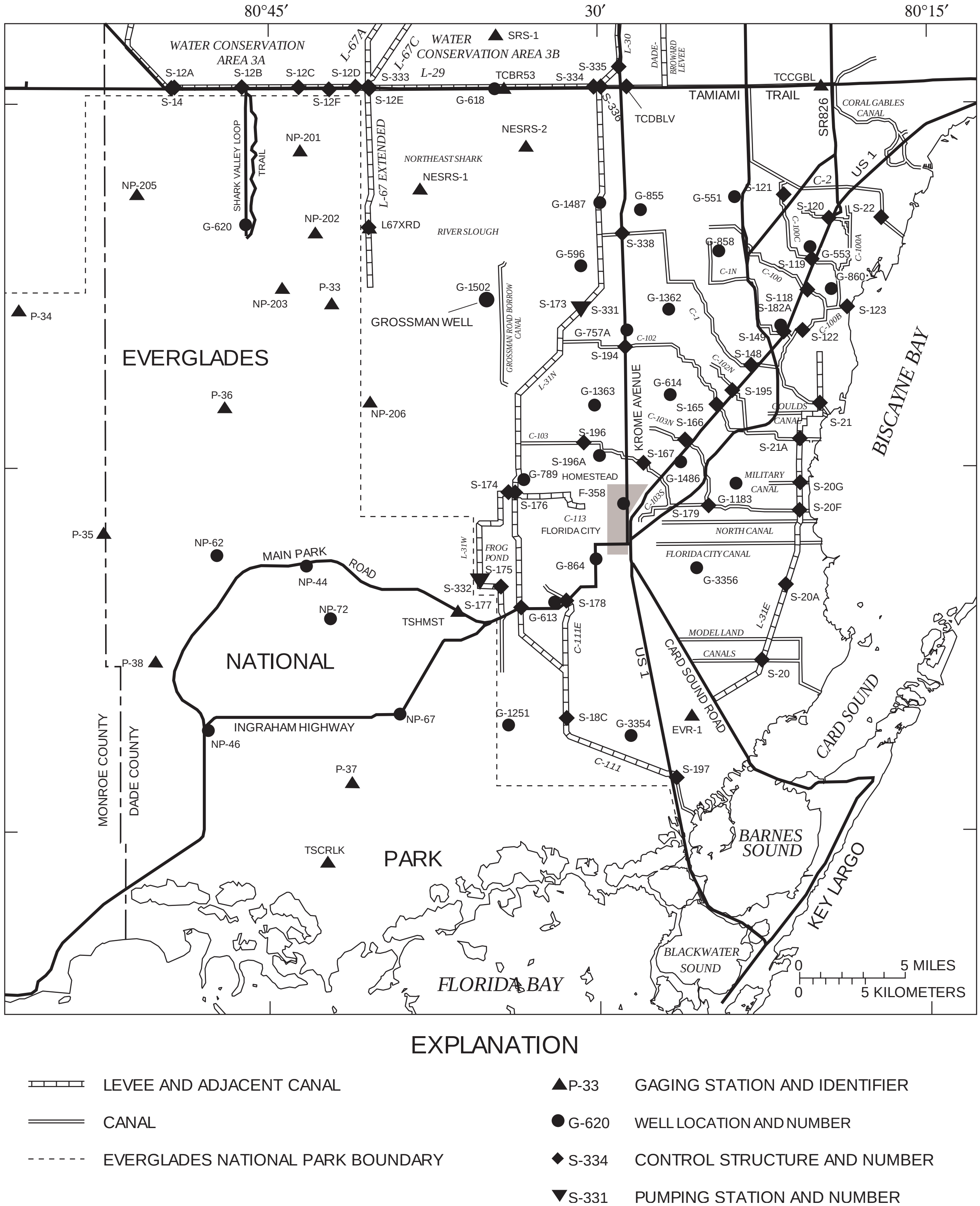


Figure 21. The canal-levee system in southern Dade County as it existed in water years 1983-89, and locations of wells and gaging stations providing continuous water-level and stage data during that period.

Table 6. Periods of manual operation of southern Dade County control structures

[Event: 1, Heavy summer rains and Tropical Storm Brenda; 2, heavy autumn rains; 3, heavy summer rains; 4, freak spring rain; 5, Hurricane David; 6, Tropical Storm Dennis; 7, “El Niño” winter rains; 8, Tropical Storm Isidore; 9, Tropical Storm Bob; 10, severe winter rain; 11, supply water to southern Dade County farming areas. Abbreviations: M, manual; A, automatic; T, telemetry, added to S-176 and S-174 (January 1988) and S-18C (October 1977); P, plug]

Dates	Event	L-31N basin		C-111 basin								Flow-divide structures			US 1 stage-divide structures								
		S-24 (M)	S-173 (M)	S-176 (A,T)	S-177 (A)	S-178 (M)	S-18C (A,T)	S-197 (M)	S-197 (P)	S-174[1] (A,T)	S-175[1] (M)	S-196 (M)	S-194 (M)	S-338[1] (M)	S-167 (A)	S-179 (A)	S-166 (A)	S-165 (A)	S-195 (M)	S-148 (A)	S-149 (M)	S-118 (M)	S-119 (A)
05/68-07/68	1			X	X	X						X						X					
09/68-10/68	2			X		X						X						X					
07/69-09/69	3	X		X		X		X															
10/69-11/69	2	X	X		X			X				X						X		X	X		
04/25/79-05/03/79	4	(2)		X	X	X	X	X				X	X	X		X		X	X				
09/02/79-09/03/79	5[3]			X	X	X	X								X	X	X	X	X		X	X	
08/80-09/80	3		X					X		X		X	X	X						X			
11/80-12/80	2		X		X	X	X	X			X												
02/19/81-02/23/81	10		X		X	X	X	X			X												
04/81	11		X		X		X					X	X		X	X	X	X					
08/18/81-08/26/81	6		(4)	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X		X	X
09/81-10/81	2			X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X		X	X
05/82-06/82	3			X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X		X	X
09/82-11/82	2			X	X	X	X	X		X	X	X	X	X	X	X	X	X	X	X		X	
01/83-04/83[5]	7			X							X	X	X	X	X	X	X	X	X	X			
06/83-09/83	3			X	X	X		X		X	X	X	X	X	X	X	X	X	X	X			
03/23/84-03/28/84	10			X	X	X		X		X	X	X	X	X		X				X			

Table 6. Periods of manual operation of southern Dade County control structures

[Event: 1, Heavy summer rains and Tropical Storm Brenda; 2, heavy autumn rains; 3, heavy summer rains; 4, freak spring rain; 5, Hurricane David; 6, Tropical Storm Dennis; 7, “El Niño” winter rains; 8, Tropical Storm Isidore; 9, Tropical Storm Bob; 10, severe winter rain; 11, supply water to southern Dade County farming areas. Abbreviations: M, manual; A, automatic; T, telemetry, added to S-176 and S-174 (January 1988) and S-18C (October 1977); P, plug]

Dates	Event	L-31N basin		C-111 basin								Flow-divide structures			US 1 stage-divide structures								
		S-24 (M)	S-173 (M)	S-176 (A,T)	S-177 (A)	S-178 (M)	S-18C (A,T)	S-197 (M)	S-197 (P)	S-174[1] (A,T)	S-175[1] (M)	S-196 (M)	S-194 (M)	S-338[1] (M)	S-167 (A)	S-179 (A)	S-166 (A)	S-165 (A)	S-195 (M)	S-148 (A)	S-149 (M)	S-118 (M)	S-119 (A)
05/84-07/84	3			X	X	X	X	X		X	X		X	X	X	X	X	X		X			
09/26/84-09/27/84	8[3]			X	X	X	X	X		X	X	X	X	X	X	(6)	X	X	X	X			
07/22/85-07/29/85[7]	9			X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X			X
06/23/86-07/17/86	3			X	X		X	X		X	X	X	X	X									
08/86-09/86	3			X	X		X	X		X	X	X	X	X									
03/87	10			X	X			X			X	X	X	X									
08/87-10/87	3			X	X					X	X	X	X		X	X	X	X		X			
06/08/88-06/22/88	3			X	X	X	X	X		X	X	X	X	X	X	X	X	X	X	X			
08/10/88-08/28/88	3			X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X			

[1]S-174 and S-175 first operated in August 1970 and S-338 first operated in March 1979.
[2]S-24 isolated and ineffective after construction and canal improvements of 1977-78.
[3]Closed when the hurricane veered away.
[4]11/81-04/83: S-173 opened after high-water events; after 04/83, S-173 used in conjunction with adjacent S-331 pumping station.
[5]04/83-01/85: S-196 and S-194 are usually open; S-338 has frequent openings; S-176, S-174, and S-177 used for agricultural purposes (Frog Pond Agreement).
[6]Put on low-range automatic.
[7]01/85-Present (1989): S-194 and S-196 are nearly always open; S-338 has frequent openings; S-176, S-174, S-175, and S-18C have frequent manual operation for agricultural purposes (Frog Pond Agreement).

The latter canal was considered to have adequatecapacity and did not need improvement. North of Tamiami Trail, the L-30 borrow canal was improved. The L-30 and L-31N borrow canals were joined, as were the sections of the Tamiami Canal east and west of this junction (fig. 18, 1977-90). Control structures in the Tamiami Canal west and east of the new junction (S-334 and S-336), and one replacing S-24B in the L-30 canal farther north (S-335), were used to manage interchanges of flow. Farther south, C-1 was completed through to the L-31N canal, and structure S-338 was added as a flow-divide structure.

The water-management system as it existed from 1983 to 1989 is shown in figure 21. Also shown are 27 wells and 23 gaging stations that provided data used in the calibration of the regional flow model.

At the junction of the Tamiami Canal and L-67A, the L-29 borrow canal and the eastern section of the Tamiami Canal were connected through a new control structure, S-333 (fig. 18). Although most of the new structures were completed in 1978 and 1979, construction problems delayed completion of S-331, a pumping station in the L-31N canal in the immediate vicinity of S-173, until late 1982. Effective use of the South Dade Conveyance System began in spring 1983. Various scenarios for the movement of large quantities of water were now available (figs. 17 and 21) as described below:

- Water from the northern and central Everglades could be released through S-333 and S-334, with S-336 closed, and moved rapidly into southern Dade County using pump station S-331.
- With S-334 closed, S-333 releases could augment southward seepage flows across Tamiami Trail into Northeast Shark River Slough between L-67A and L-30.
- By opening S-336, S-333 releases could be released to the Tamiami Canal east of L-30 (C-4) and thence into C-2 and the Coral Gables Canal (C-3) for control of coastal saltwater intrusion during dry periods.
- By opening S-336, floodwaters from the C-4 (eastern Tamiami Canal), C-3, and C-2 basins could be released westward to flow northward or southward in the L-31N or L-30 canals.
- S-335 could be operated in conjunction with other structures for transfer of flows northward or southward in the L-30 canal.
- When the new structures were closed and S-331 was not pumping or siphoning water, the water-management system had the configuration of the 1968-82 flood-control system.

About the time that S-331 was completed, the resolution of a legal case required the lowering of the water table to reduce the chance of flooding in a low-density residential area (the 8.5-mi^2 residential area) east and northeast of Chekika State Recreation Area and west of L-31N (fig. 2). This was accomplished by pumping large volumes of water through S-331 during the wet-season months (June-October). Frequent opening of the C-111 control structures (S-176, S-177, and S-18C) and the flow divide structures (S-194, S-196, and S-338) allowed the waters to be quickly dispersed toward coastal control structures. The control elevations (wet and dry settings) at these structures were also lowered.

In subsequent years, other events have determined the operational use of the South Dade Conveyance System. An agreement between water managers and agricultural interests (the Frog Pond Agreement) entailed maintaining canal stages in the C-111 basin below specified levels to lower the water table in the agricultural areas of southern Dade County. A detailed scenario that included monitoring of water levels in selected trigger wells was devised. The agreement, which began in 1985-86, required even more frequent opening of flow-divide structures (S-338, S-194, and S-196) as well as frequent manual operation of S-176, S-177, and S-18C in C-111 (table 6). Criteria for the operation of S-174 and S-175 were also changed to support the agreement. A further lowering of the control elevations of the automatic gates was part of the new water-management policy.

The National Park Service withdrew from the Frog Pond Agreement in early 1988 with the intention of requiring that scheduled canal stages be maintained and that scheduled water deliveries to Taylor Slough and C-111 be supplied. However, another legal agreement with agricultural interests required continued maintenance of low canal stages in the C-111 basin so that an experimental program of releases of water into the Northeast Shark River Slough would not cause flooding in the agricultural areas (MacVicar and Van Lent, 1984; MacVicar, 1985).

Changes in water-management policy are also evident in the more frequent manual operation of the flood-control system from 1983 to 1988 (table 6). Twelve such operations took place. On two occasions, the S-197 plug was removed. According to R. Mireau (South Florida Water Management District, oral commun., 1990), concurrent use of the low capacity culvert S-173 and the gravity-operated siphoning system of S-331 is normally adequate for winter dry-season deliveries of water to southern Dade County.

The recent (1990) construction of gated control structure G-211, just south of the confluence of the L-31N canal and C-1 (fig. 21), allows the lowering of the water table in the 8.5-mi^2 (square mile) residential area with less pumping and a reduced degree of controlled releases in the C-111 basin. The planned construction of levees around the residential area for mitigation of the effects of increased water deliveries to Northeast Shark River Slough would further reduce the need to use the South Dade Conveyance System for flood control in the residential area.

The lack of water supplied to Everglades National Park during the drought of 1962-64 led to congressional legislation in 1970 requiring minimum annual deliveries to be made to the park (Wagner and Rosendahl, 1982, p. 3). A flaw in this plan permitted the erratic scheduling of deliveries so that water-management needs elsewhere in the region could be satisfied (U.S. Army Corps of Engineers, 1992, p. 12). The ill-timed deliveries adversely affected park ecosystems, particularly in the wet spring of 1983 when the park was inundated with surplus waters from the north. The park dropped its minimum release requirement that year and began investigating alternative delivery scenarios.

A flow-through plan was devised that entailed leaving the S-12 structures fully open all year. S-12A, S-12B, and S-12C were fully open from June 1983 to June 1985, and S-12D was open from June 1984 to June 1985. In April 1984, the park began experimenting with flow restoration to Northeast Shark River Slough, but deliveries have been restricted by the previously cited need to avoid flooding in residential and agricultural parts of the area (MacVicar and Van Lent, 1984; MacVicar, 1985). In the latter part of 1985, the flow-through plan for the S-12 structures was replaced with a rain-driven plan (Neidrauer and Cooper, 1989), in which records of historical conditions and current stages in the water-conservation areas were used to determine the schedule of water deliveries to the slough east and west of L-67A. This was intended to more nearly match the natural timing of inflows to the park, while still maintaining stages in the water-conservation areas necessary for water-management purposes.

The further modifications to the water-management system that received consideration by the U.S. Army Corps of Engineers (1992) were mainly designed to restore natural flows into Northeast Shark River Slough. Most of the proposed alternatives entail the construction of water-release structures in L-67A and L-67C to establish better control of the stage in WCA-3B, and the construction of water-release structures in the L-29 seepage levee between L-67A and L-30 to increase southward flows into Northeast Shark River Slough. Other options considered were the reduction of the height of L-29 to road level and the addition of a pumping station near S-334 to discharge water from the L-31N canal into Northeast Shark River Slough. All the alternatives considered entail the removal of L-67 Extended and the filling of its borrow canal. Plans by the U.S. Army Corps of Engineers have received added impetus as a result of congressional legislation mandating the acquisition of most of the Northeast Shark River Slough for inclusion in Everglades National Park.

Water-control changes under consideration for the C-111 basin (U.S. Army Corps of Engineers, 1994) include plans for improvements to various canals and structures to permit the discharge of large quantities of water under better control through reaches of the L-31N canal and C-111 to Barnes Sound or by dispersal into the southern glades. Some of the alternatives considered would provide better management of fresh floodwater discharges to Manatee Bay (an inlet of Barnes Sound), and a reduction of their deleterious effect on estuarine ecosystems. As a first step, S-197 was enlarged in 1990 to 13 gates to allow larger controlled flood releases to the estuary that will cause less abrupt changes in water chemistry than those that have resulted from the temporary removal of the earthen plug.

Hydrologic Effects of Water-Management Changes in Southern Dade County

As explained in the previous sections, construction of canals, levees, and control structures in southern Dade County took place partly for the purpose of controlling the altitude of the water-table surface. In the following sections, the effects of construction for water management are examined qualitatively and quantitatively by the analysis of water-level and stage data collected under the auspices of the USGS and other governmental agencies.

Changes in Average Water-Table Altitude

The water-management system that has evolved in southern Dade County, beginning with the agricultural canals, has altered the typical seasonal hydrologic regime. This section of the report assesses manmade changes in typical hydrologic conditions by examining changes in long-term average water levels in selected wells and long-term average stages at a selected gaging station.

Most of the water-level and stage data used for this analysis is stored as daily values or the value every fifth day in the data base maintained by the USGS in Miami (table 2). These data have been supplemented with daily values provided by the National Park Service from data sites previously maintained by the USGS.

The daily or fifth-day values were converted to means for each month in the period of record of each station. Long-term averages for each of the 12 months of the calendar year were computed for five water-management time periods approximately as previously described: water years 1940-52, 1953-61, 1962-67, 1968-82, and 1983-89. In this way, the seasonal behavior of the water table is presented on a statistically averaged basis in each time period. The beginning of the first time period corresponds to the first available data from two of the stations (S-182A and F-358). No data before water year 1953 were available from P-35, and 2 or 3 years of water-level data before 1953 from stations G-596, G-618, and G-620 were not used because they were obtained during rainfall deficient years and might not have provided representative statistics. Locations of all wells and gaging station P-35 are shown in figures 16, 17, 19, 20, and 21.

The time-period averages (fig. 22) for G-618 indicate a marked difference between the 1953-61 time period (before construction of the seepage levee) and later time periods (after construction of the seepage levee). The difference is even more pronounced at G-596 about 8.5 mi south of the levee. Differences between the 1953-61 averages and later time periods are also evident at S-182A on the east side of the coastal ridge, and to a lesser degree, at F-358 in Homestead. This indicates that the water table throughout southeastern Dade County might have been lowered either by construction of the seepage levee and the consequent lowering of surface-water stage and water-table altitude on its south side after 1961, or by the construction of Black Creek Canal (C-1) and Belaire Canal (C-1N) in 1962. By virtue of the various well locations, G-618 water levels would show the influence of L-29 construction, whereas water levels in S-182A would be altered primarily by the construction of C-1N, which passes 0.4 mi to the south and to the west of the well. Water levels in G-596 would show the influence of both L-29 and C-1.

Average water levels during the 1940-52 time period in S-182A and F-358 corroborate the high wet-season average water levels of the 1953-61 time period. However, dry-season average water levels were as low as average water levels after 1961, indicating either that the higher average dry-season water levels of 1953-61 might have been anomalous or that the earlier dry-season water-level averages might have been lowered by the unusually low dry-season water levels of 1943-45.

The second major variation evident in the long-term averages is the raising of dry-season water levels in the coastal ridge wells and G-618 after 1982. In fact, average water levels in the 1983-89 time period show little seasonal variation in wells G-618, G-596, S-182A, and F-358. This probably results from the use of the South Dade Conveyance System, beginning in 1983, to drain residential and agricultural areas during the wet season and to supply water to southern Dade County through S-173 and S-331 during the dry season to prevent the water table from dropping to an undesirable level. The very rainy dry season of 1983-84 might have slightly contributed to the higher 7-year average.

Water levels in G-620 in the northern part of Everglades National Park and the stage at P-35 in the lower part of Shark River Slough show significant variation only during the 1962-67 time period, possibly because little or no water was supplied to the park during the drought years of 1962-64, immediately following construction of L-29. However, the similarity of average heads (water-table altitudes and surface-water stages) before 1962 (when water still flowed freely from the north through culverts in the Tamiami Trail) and in later time periods (when artificial minimum annual delivery, flow-through, and rain-driven schedules were used to supply water to the park) suggests that the imposition of controlled releases have not caused substantial variations, on the average, from heads in the park that were natural between 1953 and 1961. (Average wet-season stages may have been higher before construction of the Lake Okeechobee levee and drainage of the lake by major canals between 1909 and 1924.) Another possibility is that construction of L-67 Extended in 1967 may have counteracted the influence of L-29 in lowering average wet-season stages.

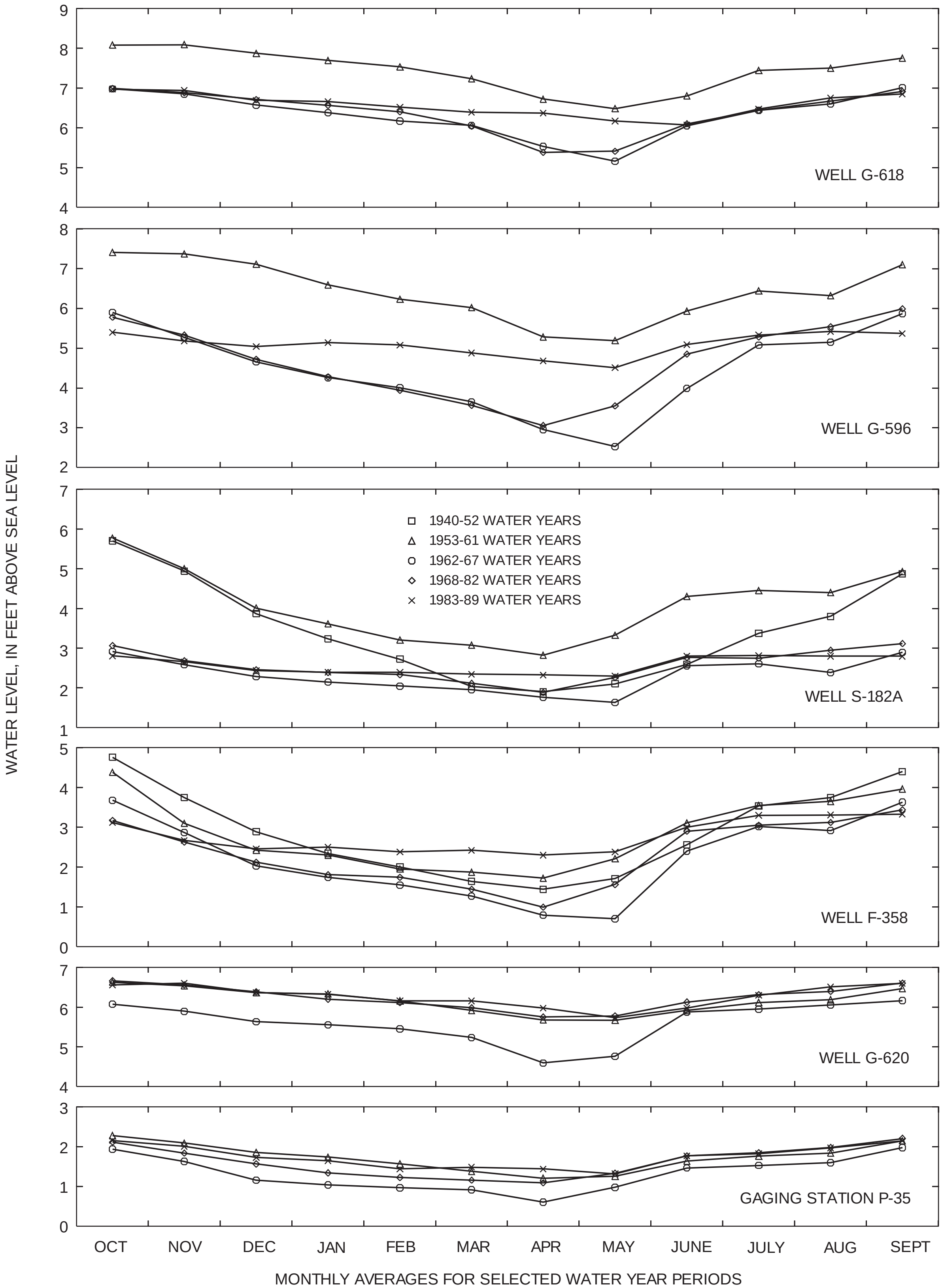


Figure 22. Monthly long-term average water levels in five selected wells and stages at a surface-water gaging station in successive water-management time periods.

Also noteworthy is the similarity of average water levels in coastal ridge wells G-596, S-182A, and F-358 during the 1962-67 and 1968-82 time periods, before and after the greatest physical expansion of the system of canals and levees in southern Dade County. The canal-levee system, however, was used primarily for flood control from 1968 to 1982, and flooding only occurred for short periods in 1968, 1969, 1981, and 1982 (table 6). Appreciable average water-level changes occurred only after 1982, with the use of pump station S-331 and inland control structures for wet-season removal of surplus water and the use of S-173 and S-331 for the dry-season replenishment of the ground-water reservoir.

Physical Description of Canals and Levees

Most canals in southern Dade County have been excavated in the coastal ridge where a thin (less than 2 ft) layer of soil covers solution-riddled limestone, or in the rocky glades (fig. 3, region 2A) where permeable limestone extends to land surface. Parts of L-31N and the Tamiami Canal were dug through a thin (less than 10 ft) layer of Everglades peat (fig. 3, region 1B), and parts of C-111 and the L-31E borrow canal were dug through thin (less than 10 ft) layers of calcitic mud in the southern and eastern glades. Therefore, except for the extreme western section of the Tamiami Canal near the Monroe County line, all canals in southern Dade County probably penetrate limestone rock having significant permeability. In the western Tamiami Canal section, limestone deposits correlative with the permeable limestone found elsewhere are thin or absent.

The physical dimensions of each canal in Dade County are known to be locally variable, but detailed data on canal depths and cross sections were not obtained for this study. The remoteness of many sections of major canals from road crossings that provide vertical access limits ability to make field measurements. The author and J.L. Labowski (formerly of the Metro-Dade Department of Environmental Resources Management) measured depths and widths of several major and secondary canals at selected locations where they were crossed by roads with the results shown below.

The Grossman Road and Krome Avenue borrow canals were obstructed by weeds in several locations. The sides of all canals were nearly vertical.

Model calibration problems led to a study of the depths of the North and Florida City Canals (fig. 21). The North Canal is 3 to 6 ft below land surface along its entire length. The Florida City Canal is 7 ft below land surface at L-31E near its ocean end, and depth below land surface decreases gradually westward until the canal bottom merges with land surface about 1.5 mi west of Florida City. (In the early decades of this century, the canal provided navigation into the heart of Florida City.) Based on this limited direct evidence and the practice in previous investigations (Camp, Dresser, and McKee, 1984), it is assumed that major canals in Dade County can be generally characterized as 100 ft wide and 15 ft deep and having nearly vertical sides. Most secondary canals probably do not differ significantly from a general description as being 5 to 6 ft deep and 10 to 20 ft wide.

Chin (1990) proposed a method for computing leakage from typical southern Dade County canals. One example selected for testing the method was the

Canal name	Location	Approximate depth (feet below land surface)	Approximate width (feet)
L-31N	SW 136th Street	22	100
	SW 168th Street	15	60
Tamiami Canal	Wooden Bridge at Coopertown (4 miles west of L-30)	15	120
North-south lateral canal	North of Coopertown Bridge	6	Unknown
Grossman Road borrow canal	Two locations near SW 168th Street	5-10	15
Krome Avenue borrow canal	Several locations	5	Unknown

upper reach of the L-31N canal, just south of the Tamiami Trail. Bottom sediments about 1.5 ft thick were cored, and the hydraulic conductivity of the sample was very low (0.03 ft/d), indicating that the bottom sediments provided some temporary confinement between the canal waters and the underlying aquifer. However, the layer of sediments was apparently not present on the sides of the channel, indicating that most leakage was out of the sides; that is, the channel was not effectively confined from the aquifer.

As is typical of the L-31N canal during the dry season, velocities in the channel were too low to be measured with the needed accuracy by standard current meters during Chin's study, and acoustic velocity meters (Laenen and Curtis, 1989) were used for that purpose. Typically, velocities in most canals in the study area can range from being too low to be measurable with any presently available instrumentation when control structures are closed to being very rapid and easily measurable when control structures are opened to release floodwaters.

The Dade-Broward Levee is an example of a road-level muck levee. These levees were constructed in the 1910's and 1920's by scooping up soft surface peats in the Everglades and using the material to form elevated banks. Over time, these levees have dried and have been subject to burning and oxidation. Sections of the Dade-Broward Levee are reported to have entirely disappeared (Isaac Sznol, Metro-Dade Department of Environmental Resources Management, oral commun., 1990). The muck levees could be overtopped by floodwaters during extreme high water events. Most Dade County levees are of more recent construction and consist of crushed limestone rock dredged from borrow ditches that penetrate the Biscayne aquifer below surface soils. They generally rise 5 to 15 ft above surrounding land surface.

Canal-Aquifer Interconnection

Characteristics of the L-31N canal reach investigated by Chin (1990) illustrate the conceptual view of canal-aquifer interaction accepted in the study as representative throughout the study area, which is that the canals are assumed to be in direct hydraulic connection with the aquifer through their sides, even though sediments providing a temporary period of confinement might line the bottom. Such a scenario might not be as appropriate in coastal regions north of the study area, where layers of calcareous sands or sandstone separate canals from underlying permeable limestones.

Evidence that would lend credence to the high degree of hydraulic interconnection, assumed to exist between canals in the study area and the Biscayne aquifer, is sought from a comparison of hydrographs of data from canal-stage recorders and wells drilled nearby. Several examples of adjacent canal gaging stations and wells providing continuous and contemporaneous records are those shown in figure 23 (site locations are shown in figs. 16, 17, 19, 20, and 21).

Figure 23 compares the hydrographs of well G-619 and the stage recorder in the Tamiami Canal below S-12B for the 1964 water year. The open-hole section of G-619 was from 6.7 to 12.7 ft below land surface. At this location, a thin layer of permeable rock occurs beneath the surface layer of peat probably to about 13 ft below land surface (Causaras, 1987). The stage recorder was in a pool of water that opens to the Tamiami Canal, and G-619 was 500 to 600 ft south of the canal and 0.4 mi east of S-12B. The G-619 data were stored for every fifth data, and the hydrograph does not have the temporal resolution of the daily values record illustrated by the S-12B hydrograph.

The comparison (fig. 23) shows that the stage in the pool south of S-12B generally remained about 0.3 ft higher than the water level in the well, and the two heads responded similarly to rainfall events and evaporation throughout a 2-ft range of variation. It was when the heads fell below land surface that the degree of hydraulic connection between canal and aquifer was revealed. Under these conditions, the canal remained wetted but was not hydraulically connected to the well site through a common pool of surface water. Atmospheric influences had similar but independent effects upon the water level in the well and the canal stage.

During the 1964 water year, Tamiami Canal stages were not affected by S-12 releases, so that the western section of the canal acted as an isolated pool linked through culverts under the old Tamiami Trail to surface water in the surrounding wetlands. From February to May, when canal stages and well water levels were below land surface, the difference between them increased to nearly 0.6 ft as the heads decreased. Rainfall events raised the water level in the well as high or higher than the canal stage. If the canal and aquifer had no hydraulic connection and received the same recharge, the head responses would have differed by the approximate 5:1 ratio of storage coefficients (storage coefficient is assumed to be 1.0 for surface-water bodies). An earlier analysis indicated that

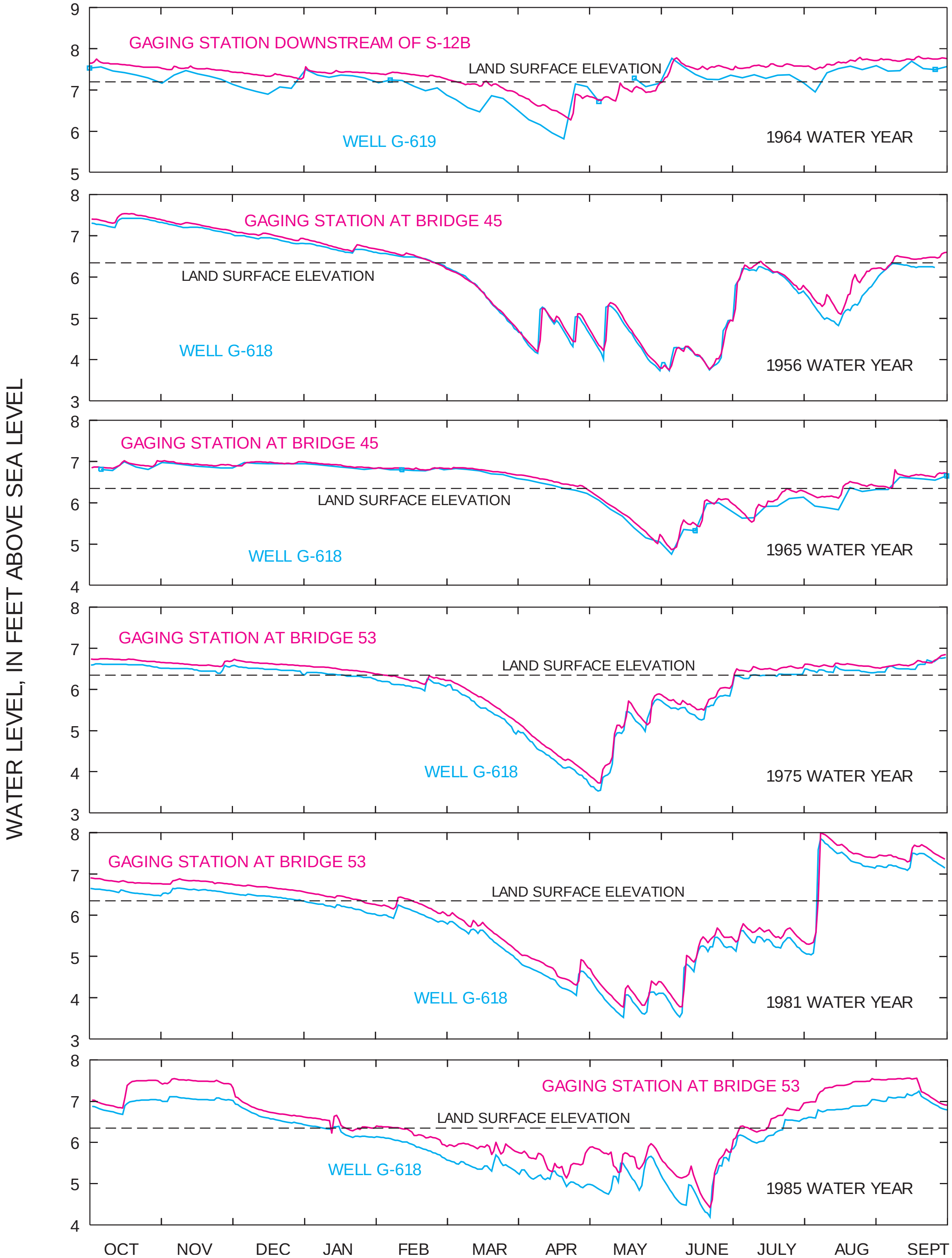


Figure 23. Canal stages and water levels in wells at selected sites in southern Dade County in selected water years.

the surface layers of peat and marl were not confining on a daily time scale. The hydrograph comparison indicates that even though head responses in the well were greater than those in the canal, the ratio is less than would occur if complete hydraulic separation were the case. The evidence indicates a weak degree of hydraulic interconnection. The factor limiting the degree of hydraulic connection at this site may have been the confinement of the aquifer by surface peats that could underlie the shallow Tamiami Canal, or the low transmissivity of the aquifer due to its thinness or lack of significant permeability in this area.

Surface-water stages and water-table altitudes are also compared in the hydrographs (fig. 23) of well G-618 (about 200 ft south of Tamiami Canal) and from the stage recorder at Bridge 45 on the Tamiami Trail (3 mi west of G-618). The comparisons are for water year 1956, when the Tamiami Canal in this vicinity was a shallow, weed-choked borrow ditch; and for water year 1965, 2 years after construction of the seepage levee, which probably deepened the original borrow ditch, but still 12 years before the 1977 canal deepening project. The open-hole section of G-618 extends from 10.5 to 20 ft below land surface and taps the Biscayne aquifer, which extends to about 35 or 40 ft below land surface at this site. All data are daily values except the 1965 water year data from G-618, which are fifth day values, and therefore, lack the detailed resolution of head changes shown by the canal stage data of the same time period.

When the two heads were below land surface in March, April, May, and June 1956, abrupt head increases caused by rainfall events were nearly identical at both recorders. Usually, the water level in the well seemed to increase first, followed 1 day later by the canal stage. This interpretation of the data is subject to the reservation that well water levels are daily highs, whereas recorded canal stages are computed as the daily mean. The comparison indicates that when the canal and aquifer received only rainfall and evapotranspiration stresses, the heads remained virtually the same and showed no differences related to differences in specific storage capacity.

A different relation is apparent in August, when the canal stage was higher than the well water level by as much as 0.8 ft, and the well water level and canal stage were no longer in close correlation. The head-relation fluctuations in August may have been caused by the numerous small storms of that month, which were likely to have differed somewhat in local intensity. The nearness of heads to land surface suggests that some source may have released surface water near the gaging station in the borrow ditch, unprotected in 1956 by a levee. It is noted that the weed-choked borrow ditch was described as nonarterial by J.H. Hartwell (U.S. Geological Survey, written commun., 1953) and stage changes in the canal near G-618 may have differed from those at Bridge 45, 3 mi away.

In the 1965 water year, the Bridge 45 recorder measured the local stage of the deepened Tamiami Canal south of the seepage levee. When the two heads were below land surface, the responses of well water levels and canal stages to rainfall events and evapotranspiration generally were similar. Slight apparent differences are partly explained by the reduced temporal resolution offered by the fifth-day values available in the G-618 data set. The canal stage usually was slightly higher, becoming as much as 0.3 ft higher in mid-August. The data indicate a relatively close hydraulic interconnection between canal and aquifer. Because the canal presumably was more arterial than in 1956, the increase of the canal stage relative to the G-618 water level in August indicates that a stress may have been applied solely to the canal cross section and that the water table did not respond rapidly to the change in canal stage.

The next three hydrographs (fig. 23) compare G-618 water levels with stages recorded at Bridge 53 on the Tamiami Trail about 0.5 mi east of G-618. The surface-water stage recorder measured stages equivalent to those in the Tamiami Canal at this site. The hydrographs for the 1975 water year show data collected before the 1977 canal deepening project, whereas the 1981 and 1985 hydrographs are from time periods after the canal was deepened and after construction of spillways S-333, S-334, and S-335, and the S-336 culvert.

During the 1975 water year, the only influxes to the Tamiami Canal section were levee seepage and rainfall. The hydrographs show close agreement when heads were below land surface. The canal stage remained about 0.15 to 0.20 ft above the water level in G-618 throughout most of the water year.

During the 1981 water year, small eastward-flowing releases through S-334 were made in March and April. S-333 releases were also made during this period from June to mid-September. The well and canal stage hydrographs show close agreement, but the head separation seems to have increased slightly to

0.25 or 0.30 ft. Apparently, structure releases were sufficiently small that L-29 and the Tamiami Canal between L-67A and L-30 continued to behave hydraulically as a seepage levee and distributor canal for southward flows as they did before the new structures were built.

During the 1985 water year, the flow-through scenario of deliveries to Everglades National Park was followed until June, when the rain-driven scenario of releases to the park and to Northeast Shark River Slough was implemented. Before June, large deliveries were made to southern Dade County by way of the South Dade Conveyance System. This entailed almost continual large releases through S-333 into the reach of the Tamiami Canal between L-67A and L-30 and through S-334 into the upper reach of the L-31N canal. S-333 and S-334 were both closed before October 15, 1984, in December 1984 and early January 1985, from mid-June to mid-July 1985, and after September 18, 1985. During these periods, the canal stage and aquifer water-table altitude maintained the equilibrium relation noted in the earlier water years. When large S-333 and S-334 releases occurred, however, that relation was somewhat disturbed, although heads never differed by more than 0.6 ft, and both heads clearly responded similarly to climatic stresses when they were below land surface.

Though a hydraulic connection is evident, water levels in G-618 and canal stages did not maintain a stage relation as uniform in the 1985 water year as in the 1975 and 1981 water years. The conditions that differed in the 1985 water year from those of the earlier water years were that canal-stage changes resulted from S-333 and S-334 releases as well as from local rainfall events and seepage under the levee. S-333 and S-334 releases caused rapid stage changes that were confined to the canal cross section until leakage occurred, whereas atmospheric stresses were areally distributed and influenced the water table and canal stages more gradually and simultaneously. Structure releases, therefore, specifically stress the leakage capacity of the canal. The head differences shown in parts of the 1985 water-year comparison might indicate: (1) the limited ability of the aquifer to respond to rapid canal-stage changes, or (2) that some degree of canal-wall clogging reduces the degree of connection between canal and aquifer.

Figure 24 shows head relations in water year 1985 between well G-1487, the stage downstream of structure S-334, and the stage upstream of S-331 (locations shown in fig. 21). G-1487 is nearly midway between the two structures and about 100 to 150 ft west of the canal. The casing depth is unknown, but the total well depth is 20 ft. At this site, rocks of the Biscayne aquifer extend from below a surface layer of peat to 50 ft below land surface. The reach of the L-31N canal between S-334 and S-331 is not blocked by any structures and was improved in 1977.

Canal stages and the water table are both below average land surface (about 7 ft) throughout the 1985 water year. The schedule of S-334 releases was previously explained, and S-336 releases were made from February to June. Substantial releases were made through S-335 from mid-December to mid-March. The effect is clearly evident in separating S-334 and S-331 stages by about 0.4 ft. However, canal stages and aquifer water-table altitudes are apparently in close correlation at all times. During the dry season, from October to late June, G-1487 water levels are nearly identical to stages at one or the other of the canal recorders. During the following wet season, well water levels rise above the S-334 stage during rainfall events but fall back nearly to the S-334 stage between events. The record demonstrates significant hydraulic interconnection between the L-31N canal and the aquifer in the immediate vicinity, but also shows that abrupt climatic stresses and structure releases can cause slight head separations.

These comparisons have shown that canal stages and water-table altitudes in nearby sections of the Biscayne aquifer are similar, and their variations are closely correlated. Subtle differences have been associated with either evident or hypothetical causal factors that suggest some limitations in the degree of canal-aquifer hydraulic connection. However, in construction of a regional model of flows in the Biscayne aquifer of southern Dade County, canal stages can readily be identified with local aquifer water-table altitude as an adequate approximation for specification of boundary conditions or for use in calibrating the model. Additionally, the representation of canal reaches in the model should allow sufficient leakage across the boundary with the aquifer that heads remain nearly the same under all hydrologic conditions.

Despite these results, it should be noted that either the aquifer water-table altitude near a canal or the canal stage (or both) may be considerably modified by recharge to the aquifer from the canal or by discharge to the canal from the aquifer. The local water-table altitude might be significantly different if an existing canal were not present.

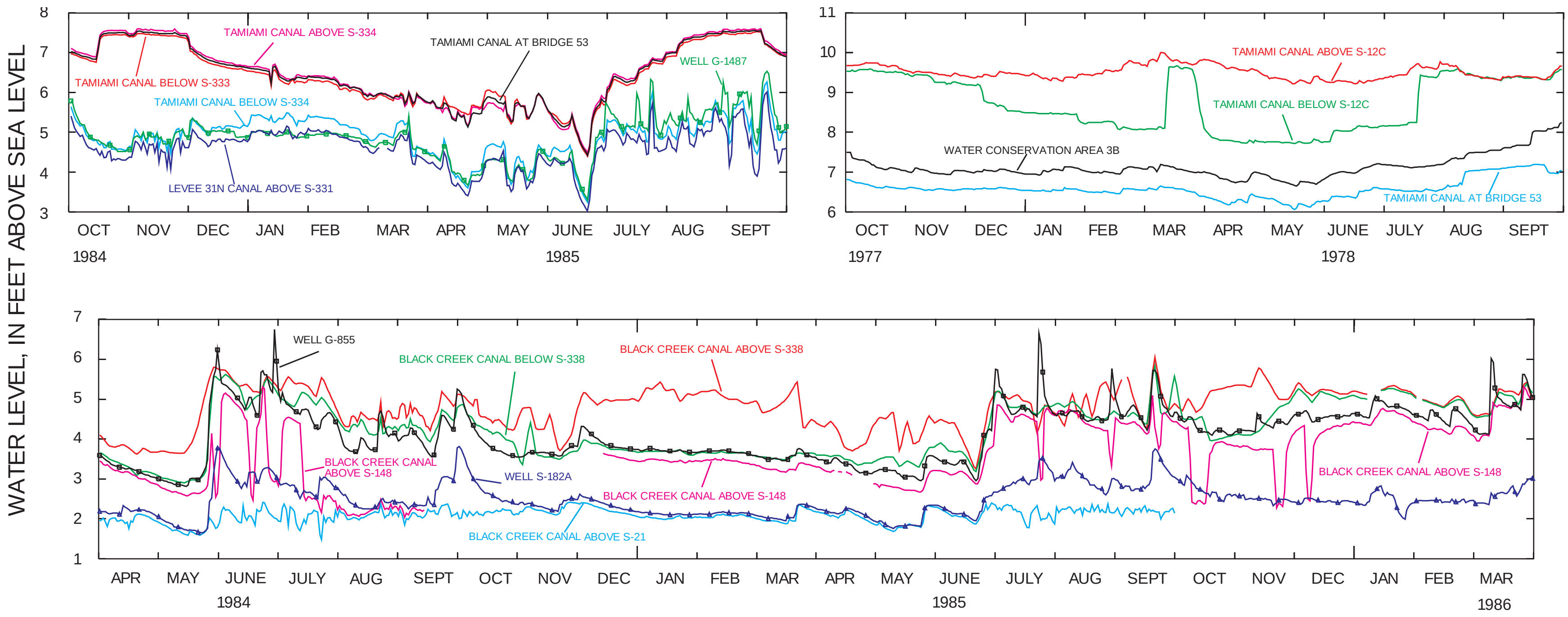


Figure 24. Surface-water stages upstream and downstream of selected canal control structures, upstream and downstream of Levee 29, and water levels in selected wells in selected water years.

Hydraulic Influence of Canals

It is of significant interest to compare stage differences in reaches of Dade County canals with head differences in wells separated by comparable distances. The insight gained into the regional hydraulic influence of the canals upon the water table helps in understanding the problems related to construction of a regional flow model.

Figure 24 includes 1985 water-year hydrographs of stages within Tamiami Canal and the L-31N canal and water levels recorded in well G-1487. The canal-stage locations are, from west to east and then south: downstream (east) of S-333, at Bridge 53, upstream (west) of S-334, downstream (east) of S-334, and upstream (northeast) of S-331. (Tables 2 and 3 provide site data, and figure 21 shows the site locations.) S-333 and S-334 are about 11 mi apart, and Bridge 53 is about 4 mi west of S-334. The reach is continuous without structures or dams. As noted, major releases through S-333 and S-334 stressed the stage relations among the various recorder station in water year 1985. Stages downstream of S-333 and upstream of S-334 each varied by more than 3 ft and often abruptly, as in June. Except for short periods in April, May, and June, the two stages never differed by more than 0.15 ft, and the maximum difference was about 0.3 ft. Usually the stage upstream of S-334 was higher, indicating westward flow and drainage into Northeast Shark River Slough, but for short periods in April, May, and June, the stage downstream of S-333 was higher. These latter periods correlate well with exceptionally large S-333 releases. The stage at Bridge 53 was intermediate throughout the entire year.

The stages downstream of S-334 and upstream of S-331, about 11 mi apart in an unimpeded canal reach, each varied more than 3 ft, but the stage downstream of S-334 was nearly always 0.2 to 0.3 ft higher than that upstream of S-331. However, the difference in stage across structure S-334, a distance of a mere few feet, was as high as 3 ft and was almost never less than 1 ft in the 1985 water year.

Water-level contour maps of Dade County prepared by R.S. Sonenshein (U.S. Geological Survey, written commun., 1985) show the water-table altitude in 1985 decreasing approximately linearly about 2 to 3 ft in a southeasterly direction, from S-333 to S-331. In 1985, in the 22 mi of Tamiami Canal and the L-31N canal reaches between S-333 and S-331, only about 0.2 to 0.5 ft of head loss occurred within unimpeded reaches of the canals, whereas virtually all of the 2 to 3 ft of head loss occurred at the closed control structure. Evidently, with increasing distance from the structure, the water-table altitude grades from the abrupt discontinuity at the closed structure to the more linear variation shown on the contour maps.

Unimpeded reaches of canals, therefore, seem to act as “short circuits” in the upper part of the Biscayne aquifer, given their close hydraulic connection with the aquifer. There is probably a considerable degree of ground-water seepage around the structures when they are closed and maintaining an appreciable difference in stage. The structures can probably maintain appreciable stage differences because of the substantially lower resistance to open-channel flow than to flow within an aquifer, even one as permeable as the Biscayne aquifer. A contributing factor of unknown relative significance may be the restriction of flow through the sides of the canals by sedimentation or clogging.

The complex relation of stages in canal reaches and at control structures to heads in the aquifer is even more evident in the hydrographs (fig. 24) showing stages and water levels in the Black Creek Canal (C-1) basin from April 1984 to March 1986. Stages shown, in downstream order, are upstream and downstream of S-338 (near L-31N), upstream of S-148, and upstream of S-21 (the coastal control structure). Contemporaneous water levels are shown in hydrographs of well G-855 (1.5 mi northeast of S-338) and of well S-182A (2.5 mi northeast of S-148). G-855 is about 1.2 mi north of C-1 and about 2 mi east of the L-31N canal, and S-182A is 0.4 mi north of control structure S-149 on C-1N and 0.4 mi east of another section of C-1N (site locations and data are presented in fig. 21 and tables 2 and 3).

Stages upstream and downstream of S-338 differed by as much as 1.5 ft when the structure was closed. From mid-August to late October 1984, the structure was partly open but still maintained a head difference of about 0.3 ft between upper and lower pools. When S-338 was more fully open, as in late November 1985 to March 1986, the stage upstream of the structure was no more than 0.1 ft higher than that on the downstream side. S-338 and S-148 are 10.3 mi apart, yet the unimpeded intervening reach between them usually separated the stage downstream of S-338 from the stage upstream of S-148 by only about 0.3 ft. Exceptions occurred during manual openings of S-148, usually of 3 to 5 days duration in the time period shown. The stage effects are shown in figure 24 as downward spikes in the hydrograph of S-148 in May and June 1984 and in July to December 1985. Upward spikes in the hydrograph of G-855 show that the manual openings coincided with intense rainfall events, and simultaneous downward spikes in the stage upstream of S-338 in July and August 1985 show occasions when S-338 was also opened in response to rainfall.

The recorded water level at G-855, which generally represents the water-table altitude in a larger area encompassing S-338, remains close to the C-1 stage south of S-338, indicating that when control structures are closed, the downstream stage is representative of the local water-table altitudes. Thus, in January and February 1985, the stage upstream of S-338 is about 1.5 ft above the local water table. The reach of C-1 upstream of S-338 is open to a reach of the L-31N canal that extends north to S-334, so that the stage upstream of S-338 probably is representative of a water-table altitude much farther upgradient. In fact, the stage upstream of S-338 usually is intermediate between that downstream of S-334 and that upstream of S-331, and is similar to that at well G-1487.

From mid-July to October 1984, S-148 was operated manually in a fully open position while S-338 was partly open. During at least the first part of this period, the stage upstream of S-148 closely approximated that upstream of S-21, the coastal control structure about 4.6 mi downstream of S-148. (S-148 stage data from mid-September to mid-December 1984 are not available.) The stage downstream of S-338 averaged 0.3 ft lower than that upstream of S-338 and averaged about 2 ft higher than the stage upstream of S-148, indicating that substantial flow occurred in the unimpeded intervening reach. The average estimated discharge through S-21 was more than 500 ft^3/s (cubic feet per second) during this 100-day period.

When structure S-149 in the C-1N canal is closed, the downstream stage should resemble that upstream of S-21 if there are no abrupt lowerings of the latter stage. The hydrograph of well S-182A, which is near S-149, shows that the water level in the well is similar to the stage upstream of S-21 for most of the dry seasons of 1984 and 1985, when S-21 releases were negligible. (Stage data upstream of S-21 for several months after September 1985 are not available.) In the wet seasons, rainfall helped to raise the S-182A water level as much as 1.5 ft higher than the stage upstream of S-21 (S-149 remained closed). Releases through S-21 during these periods also helped to lower the canal stage with respect to the well water level. The nearness of the canal substantially affects wet-season water levels in S-182A, as was shown earlier (fig. 22) in time-period hydrographs for water years 1962 through 1989.

A general description of canal stages and their relation to the water-table altitude is evident from the examples considered. If a hypothetical southern Dade County canal is considered as being segmented into a series of reaches by closed control structures, and as generally extending from an area of higher water table to one of lower water table, the following statements can be made when rainfall amounts are not large (winter dry season):

- Stage variations within unimpeded reaches between closed structures will not exceed a few tenths of a foot, although stage might drop several feet at the structures.
- The canal stage downstream of a structure will be similar to the water-table altitude characteristic of the local area, but the canal stage upstream of the structure will be similar to the water-table altitude in the general vicinity of the next upstream control structure.
- Recharge to ground water occurs upstream of each control structure, and canal water lost to the aquifer is quickly replenished by canal flows from upstream.
- Drainage from the aquifer upstream and seepage around the upstream structure supply canal flows that recharge the aquifer downstream.
- A canal stage in equilibrium with the aquifer water table upstream where the canal drains the aquifer, but not downstream where the canal recharges the aquifer, might indicate some clogging of the sides of the canal in the downstream reach. Otherwise, the canal stage should be intermediate between upstream and downstream water-table altitudes.

When a control structure is opened, the stage below an upstream control structure that remains closed is lowered, substantial canal flow occurs, and the aquifer is drained at a more rapid rate near the upstream control structure.

Hydraulic Influence of Levees

When closed, canal control structures lower stages in narrow channels cut below land surface, permitting flows only to occur as ground-water seepage around the structures. The function of levees is similar, except that sheetflow broadly dispersed above land surface is confined by raising a barrier above land surface that transects the area where the overland flow occurs. Movement of water past a levee is by seepage through or underneath the levee or by releases through control structures. Hydrographs shown in figure 24 demonstrate the ability of levees to separate the stage of overland surface flows in the pools upstream and downstream of the levee.

At S-12C (figs. 19-21), one of the water-release structures in the western section of L-29 used to supply water to Shark River Slough, a thin layer of peat underlies surface waters, and geologic conditions are as previously described for the G-619 and S-12B sites. Average land surface is less than 7 ft at S-12C. The relatively low transmissivity of the subsurface and the slight degree of confinement provided by the peat soil contribute to the ability of the levee to separate surface-water stages to the degree shown by the 1978 water-year hydrograph of stages upstream and downstream of S-12C (fig. 24). The gates were gradually lowered in December 1977, until only small quantities of water were released. The stage difference increased to nearly 2 ft. Larger openings in March and July 1978 allowed the lower stage to rise nearly as high as the upper stage. In August, all gates were fully opened, and no stage difference is discernible.

In the 1978 water year, the stage in WCA-3B farther to the east was about 2 ft less than that of WCA-3A as a result of confinement by seepage levees L-67A and L-67C. The stage in Water Conservation Area 3B (fig. 24) was recorded at surface-water station Shark River Slough No. 1 (figs. 20 and 21, site SRS-1). The stage of water flowing southward in the region was about 0.5 ft lower south of seepage levee L-29 than at SRS-1 during most of the 1978 water year, as shown by the stage record at Bridge 53 (figs. 20 and 21, site TCBR53). The stage difference maintained by this eastern section of L-29 is much less than that maintained by the western section when some gates in the latter were not fully closed (December 1977 to March 1978 and April to July 1978). The difference probably results from the higher transmissivity of the underlying Biscayne aquifer in the eastern section (fig. 6) relative to contemporaneous deposits underlying the western section, which would permit a higher rate of underseepage in the eastern section.

All other levee sections in southern Dade County overlie highly permeable sections of the Biscayne aquifer. In the C-111 basin, levees are in regions that have recurring surface-water flow over a surface of calcitic muds, as is also true of L-31E to the east. The section of L-31N north of well G-1487 also is in a region of recurring surface flows over a peat surface. The lower section of L-31N is built in the eastern rocky glades. Since 1961, when L-29 was constructed, this levee section, which is for flood control in extreme events and has been breached for long intervals of time at some locations, has rarely needed use for confinement of surface flows, as overland flow stages have been shallow, infrequent, and of short duration.

Well Fields

The largest major well fields in Dade County are the Hialeah/Preston/Miami Springs Well Field complex, the Northwest Well Field (fig. 1), and the Alexander Orr/Snapper Creek/Southwest Well Field complex. The first two well fields in northeastern Dade County supply the Hialeah and John E. Preston Water Treatment Plants. Both well fields are sufficiently far north of the Tamiami Trail that their hydraulic influence is negligible in the study area south of Tamiami Trail. In 1987, the maximum permitted combined average capacity of the two pumping centers was 165 Mgal/d (million gallons per day). The third well-field complex in the northeastern part of the study area supplies the Alexander Orr Water Treatment Plant. In 1987, its maximum permitted average capacity was also 165 Mgal/d.

The Alexander Orr Water Treatment Plant began supplying water to southern Dade County in the 1949-50 pumping year (July 1949 to June 1950). The supply initially was from seven wells in the Alexander Orr Well Field, until six of the Southwest Well Field production wells were drilled in the 1952-53 pumping year and the major part of the pumping was shifted to the new wells. Four additional wells were drilled in the Southwest Well Field in the 1959-60 pumping year and another four wells were drilled in 1984. Four additional wells were drilled in the Alexander Orr Well Field in the 1964-65 pumping year. The four-well Snapper Creek Well Field opened in the 1978-79 pumping year. Pumping data have not been compiled for individual well fields within the complex. If needed, such data could be estimated from lists of hours of operation of the pumping station at each well and the estimated capacity of the pumps. Average daily flow leaving the Alexander Orr Water Treatment Plant was 0.53 Mgal/d in 1960-61, 68.55 Mgal/d in 1971-72, and 122.76 Mgal/d in 1983-84. Pumping data available in 1990 indicate that 144.0 Mgal/d were produced in 1987-88.

Unpublished potentiometric surface maps of Dade County prepared one or more times annually by the USGS generally show that the water table is drawn down more than 1 ft in an area about the Southwest Well Field that extends less than 0.5 mi southeast and northwest of the line of wells. Drawdown contours of 1 ft around the Alexander Orr Well Field, located farther to the east, include a larger area because the Biscayne aquifer is less transmissive in that area. Drawdowns of 1 ft around the Snapper Creek Well Field are highly localized. Drawdowns of 0.5 and 0.25 ft about the three well fields would include a much larger area. The area affected by this large volume of pumping is extensive, but the high transmissivity of the aquifer confines large drawdowns to a small area.

The second largest of the smaller pumping centers in the Homestead area (fig. 2) is the City of Homestead Well Field. Homestead had at least one pumping well in 1923, two wells 1.5 mi apart in 1946 when 0.8 Mgal/d were pumped, and in 1990 had a well field consisting of six wells at three locations within a 2.5- to 3-mi radius within the city limits. The combined average daily pumpage in 1986 was 6.3 Mgal/d and has since decreased to about 5.3 Mgal/d.

Because of the high transmissivity of the Biscayne aquifer in the southern part of the study area, the hydraulic influence of these small well fields is negligible on a regional scale. For example, two wells (G-864 and G-864A) were drilled in 1959 to observe the effects of pumping at the largest of the well fields in the Homestead area, the Florida Keys Aqueduct Authority supply well field southwest of Homestead. The maximum permitted average daily production in 1987 was 14 Mgal/d. G-864A (figs. 17 and 19, 20, and 21) is close to the pumping wells and G-864 is outside the plant property, 0.5 mi downgradient of the pumping wells. Both observation wells were equipped with digital recorders. Recent data indicate that water levels in downgradient well G-864 (figs. 17, 19, 20, and 21) tend to be 0.05 to 0.1 ft higher than those in G-864A within the well field; thus, the well-field drawdown is insignificant and highly localized.

Even smaller well fields are located along an axis drawn between the Alexander Orr Well Field and the City of Homestead Well Field. Most were originally constructed to supply local private utilities, but have since been purchased and operated as backup supply wells by the Miami-Dade Water and Sewer Authority Department. In the early 1990’s, they were largely phased out of service.

Atmospheric Influences

The recharge of the Biscayne aquifer by rainfall, most of which is subsequently released from aquifer storage by evapotranspiration, is a highly significant element of the water budget of the aquifer. To ensure that a model of the aquifer is constructed that has the needed accuracy, data describing rates of precipitation are reviewed in the following sections to determine seasonal changes and whether or not spatial variations or long-term trends are indicated. Factors determining the evapotranspiration rate are also described.

Previous rainfall studies have focused on geographic regions larger than the present study area and have been based on data from a few widely scattered stations. The following review focuses specifically on southern Dade County. Data collected in recent years from a relatively dense network of rainfall stations recently installed by the South Florida Water Management District and the National Park Service are analyzed for spatial variations.

Rainfall

Rainfall is the principal source of recharge to the Biscayne aquifer of Dade County and to the body of surface water that occurs seasonally in the county. Attempts to model Biscayne aquifer flows depend on the ability to quantify precipitation, and to identify its average seasonal variation and any significant spatial variations in the area of interest that seem to have remained consistent for significant periods of time.

The average rainfall rate was 59 in/yr (inches per year) in southeastern Florida, based on a 20-year average of data from 11 long-term index stations in Dade, Broward, Palm Beach, and Martin Counties (fig. 1) (Leach and others, 1972). MacVicar (1983) generalized the areal distribution of rainfall in the area comprising the South Florida Water Management District, based on data available at that time from widely scattered locations. The South Florida Water Management District includes all counties of southern

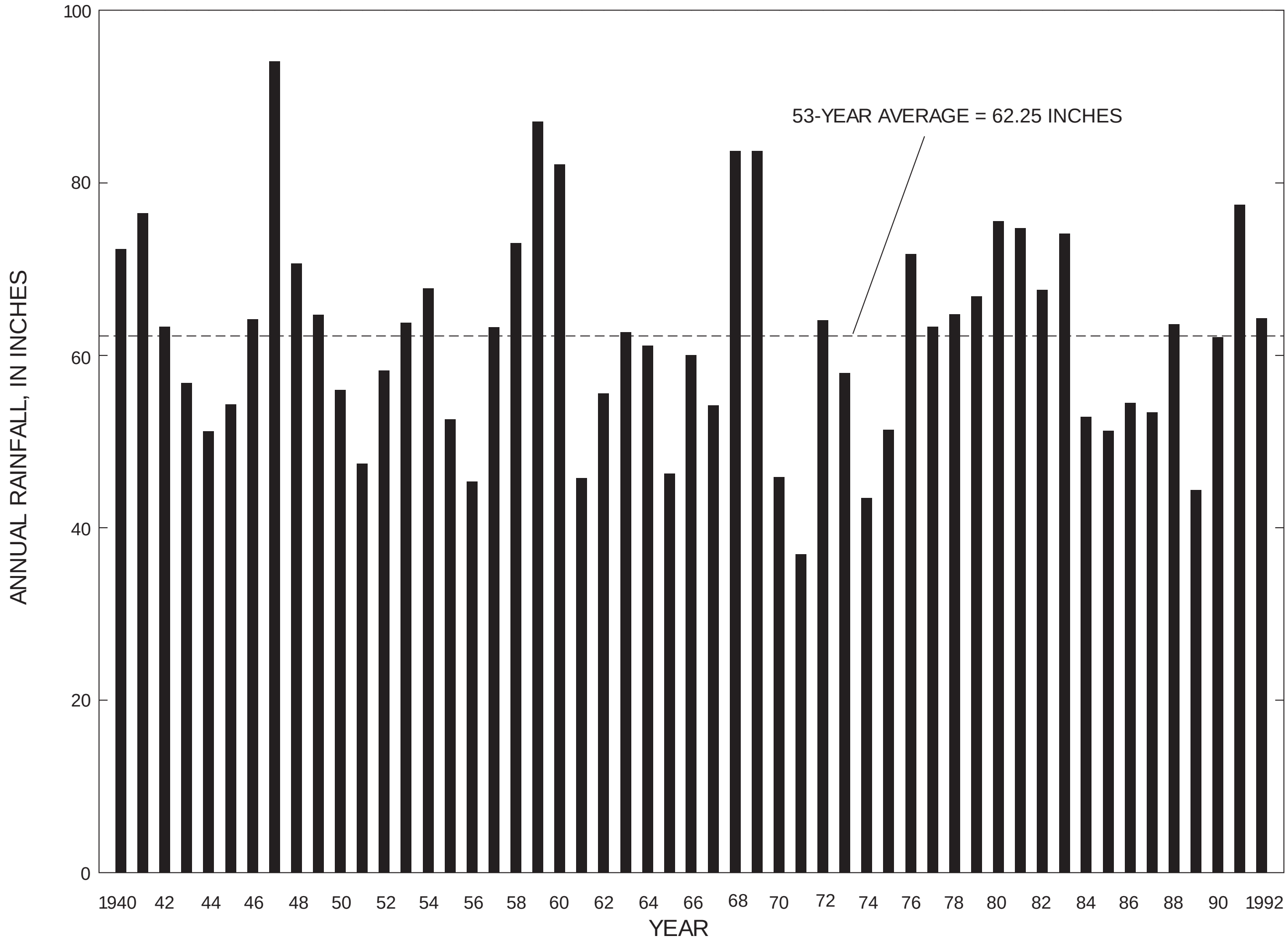


Figure 25. Annual rainfall in southern Dade County, 1940-92 calendar years.

Florida shown in figure 1 and additional areas farther north along the east coast and in the Kissimmee River Basin area of central Florida. Results of the study indicated that rainfall totals in the South Florida Water Management District area were greatest at a station near Homestead. Annual rainfall totals at this station from 1940 to 1981 and from a nearby station from 1982 to 1992 are depicted in figure 25. The 1992 total uses August, September, and October values determined by the South Florida Water Management District to be an average for eastern Dade County. Southern Dade County rainfall recorders were inoperative for a period following Hurricane Andrew in August. Individual annual totals from Homestead have ranged from a minimum of 37.0 in. in 1971 to a maximum of 94.1 in. in 1947. Periods of above-average rainfall (wet years) and of below-average rainfall (dry years) generally seem to occur in 2 to 4 consecutive years.

The data indicated that 75 percent of areal rainfall occurs in the summer when precipitation typically is in the form of local thunderstorms that may be intense in one area and virtually absent in others. Even when summer rainfall is widely distributed as the result of a trough, wave, or tropical storm, the local variation in intensity may be large. For this reason, the recorded precipitation at a measuring site during the summer months can be misleading as an indicator of the average rainfall occurring over a larger area. Therefore, use of data from many sites to prepare totals or statistical analyses is desirable when possible.

Locations and period of record of the rainfall sites providing data used in this study, which comprise a large part of the total data base available, are given in table 7, and the areal distribution of the stations is shown in figure 26. Only four sites in the study area provide rainfall data throughout most of the period of the regional flow simulation (water years 1945-89). Of these four rainfall stations, the one at Forty-Mile Bend in the northwestern part of the study area can be considered generally representative of rainfall amounts in the area in which Shark River Slough (fig. 3) borders

Table 7. Locations and period of record of rainfall data sites used in the study

[Locations shown in figure 26. Period of record: 1990+ indicates that data collection was continuing at the time of the first draft of this report (1990). Agency: USWB, U.S. Weather Bureau; SFWMD, South Florida Water Management District; NPS, National Park Service]

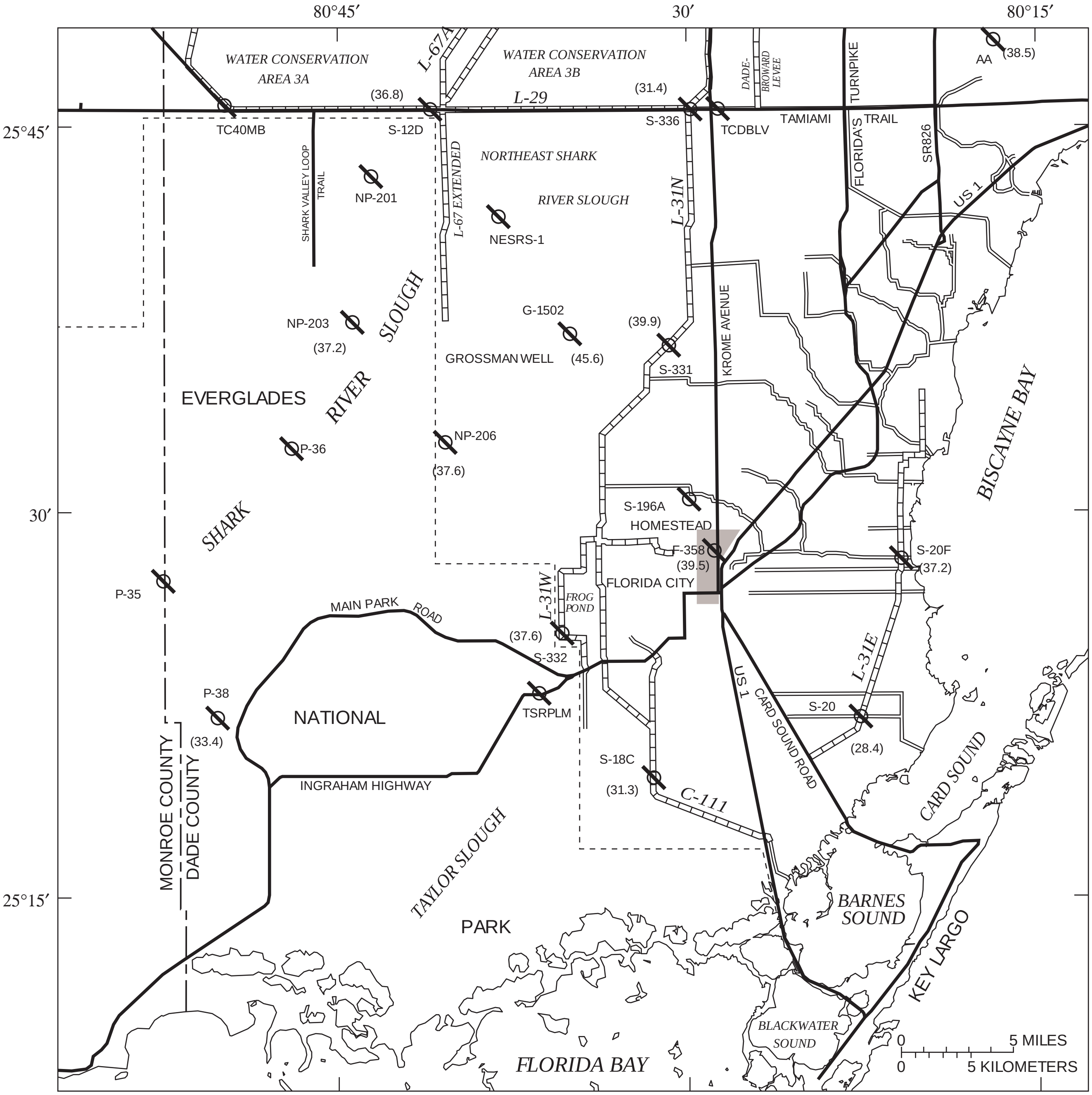
Location	Map identification code (see fig. 26)[1]	Period of record	Agency
Miami International Airport	AA	01/12-1990+	USWB
Homestead Agricultural Experiment Station	S-196A	01/31-1990+	USWB
Tamiami Trail at Forty-Mile Bend	TC40MB	01/41-1990+	USWB
Tamiami Trail at Dade-Broward Levee/Trail Glades Range entrance	TCDBLV	01/41-1990+	USWB
Royal Palm Ranger Station	TSRPLM	05/49-1990+	USWB
S-12D spillway on L-29	S-12D	01/62-10/77 03/85-1990+	SFWMD
C-111 below S-18C	S-18C	03/67-1990+	SFWMD
Homestead Field Station	F-358	01/68-1990+	SFWMD
S-20F spillway on C-103	S-20F	05/68-1990+	SFWMD
S-20F spillway on C-107 at L-31E	S-20	05/68-1990+	SFWMD
S-336 culvert on Tamiami Canal east of L-31N	S-336	05/79-1990+	SFWMD
S-331 pump station on L-31N borrow canal	S-331	08/80-1990+	SFWMD
S-332 pump station on L-31W borrow canal	S-332	04/81-1990+	SFWMD
NP-201	NP-201	10/83-1990+	NPS
P-35	P-35	10/83-1990+	NPS
P-36	P-36	10/83-1990+	NPS
P-38	P-38	10/83-1990+	NPS
NP-203	NP-203	01/84-1990+	NPS
Chekika State Recreation Area	G-1502	04/84-1990+	SFWMD
NP-206	NP-206	10/84-1990+	NPS
Northeast Shark River Slough No. 1	NESRS-1	11/85-1990+	NPS

[1]Rainfall stations, with the exception of the Miami International Airport station, are at or near the location of wells or surface-water stations listed in table 2 and shown in figures 16, 17, 19, 20, and 21.

the Big Cypress Swamp to its northwest (fig. 1). The Homestead Agricultural Experiment Station in agricultural lands of the coastal ridge and in the center of the study area seems to be ideally situated to provide data representative of the most important part of the study area. The Royal Palm Ranger Station in Taylor Slough southwest of the Homestead site (fig. 3) also provides data representative of southern coastal ridge rainfall, and the data can be correlated with values from the Homestead station when summer monthly totals at the latter seem to be unrepresentative. The rainfall station at the Dade-Broward Levee on the Tamiami Trail is on the northern boundary of the study area in Northeast Shark River Slough (figs. 20 and 21). The station was relocated to the Trail Glades Range entrance in 1966, and usefulness of the record from either location is diminished by many periods of missing data. Although not in the study area, rainfall data from Miami International Airport are used as generally representative of the coastal ridge for correlation with the Homestead data.

To assess the rainfall data available in or near the study area for trends that would reveal spatial and long-term temporal variations and to study seasonal variation, data from the five long-term sites were used to prepare averages for the five water-management time periods previously identified (water years 1945-52, 1953-61, 1962-67, 1968-82, and 1983-89) and for the entire simulation time period (water years 1945-89). Time-period averages were computed for each month of the calendar year and for the entire year. Results are given in table 8.

Comparing annual totals at the four sites in the study area in each time period suggests a spatial pattern though the data are insufficient for a definite conclusion. In nearly every time period, the lowest annual rainfall is reported from the Dade-Broward Levee/Trail Glades Range station. These values might be misleading because rainfall data from many periods of



EXPLANATION

LEVEE AND ADJACENT CANAL

CANAL

EVERGLADES NATIONAL PARK BOUNDARY

G-1502 (45.6) RAINFALL STATION AND LOCATION SYMBOL—number in parentheses is 1985-89 average total summer rainfall, in inches

NOTE: SYMBOL LOCATIONS FOR RAINFALL STATIONS (TABLE 7) OTHER THAN AA ARE WELLS AND GAGING STATIONS FOR WHICH DATA ARE GIVEN IN TABLE 2.

Figure 26. Locations of stations in southern Dade County providing data for analysis of regional variations in summer rainfall, water years 1983-89.

Table 8. Long-term monthly and annual average total rainfall at selected measurement stations

[Rainfall data shown in inches. Dashes (—) indicate data not available]

Water years	Oct	Nov	Dec	Jan	Feb	March	Apr	May	June	July	Aug	Sept	Total
Miami International Airport													
1945-52	8.32	1.94	1.83	1.13	1.46	1.92	3.79	3.64	5.27	6.21	6.35	8.63	50.49
1953-61	8.38	3.39	1.97	2.48	1.87	2.03	3.55	7.83	8.76	6.40	6.84	9.49	62.99
1962-67	7.88	2.59	2.03	1.88	2.74	2.47	1.33	3.26	11.92	5.27	7.22	8.62	57.21
1968-82	5.76	2.45	1.65	1.96	1.98	1.77	4.06	7.81	8.88	5.40	7.58	8.01	57.32
1983-89	3.82	3.56	2.31	2.05	2.07	3.71	2.09	5.01	8.28	7.46	7.00	7.14	54.50
1945-89	6.66	2.74	1.90	1.92	1.98	2.25	3.24	6.03	8.53	6.05	7.08	8.36	56.74
Forty-Mile Bend													
1945-52	7.62	1.50	1.19	.95	1.26	1.56	3.11	4.74	6.88	9.95	8.34	10.27	57.37
1953-61	5.38	1.41	1.09	1.71	1.36	2.44	2.97	6.39	8.96	7.93	6.72	9.79	56.15
1962-67	4.88	1.37	1.01	1.82	1.84	2.02	1.78	5.18	11.50	6.43	7.62	9.89	55.34
1968-82	4.21	1.84	1.25	1.59	1.61	1.35	2.36	6.00	9.83	7.46	6.84	6.69	51.03
1983-89	2.42	2.32	1.66	2.08	1.95	3.19	1.16	3.78	8.54	7.93	5.96	5.60	46.58
1945-89	4.81	1.69	1.23	1.61	1.58	1.99	2.37	5.40	9.15	7.89	7.08	8.20	53.00
Homestead Agricultural Experiment Station													
1945-52	10.03	2.16	1.28	1.19	1.41	2.00	3.03	5.28	7.37	9.81	8.59	10.72	62.87
1953-61	8.42	2.40	1.44	1.90	2.04	2.75	3.93	7.99	9.78	8.24	7.59	8.48	64.96
1962-67	6.81	1.55	.92	1.94	1.65	2.44	1.80	3.86	12.36	5.86	6.04	10.92	56.15
1968-82	5.81	2.08	1.58	1.53	2.34	1.33	2.94	7.72	10.47	6.98	10.13	9.57	62.48
1983-89	4.23	3.16	1.74	2.45	2.05	3.34	2.23	5.41	7.83	8.04	7.59	7.88	55.95
1945-89	6.97	2.26	1.44	1.73	1.97	2.20	2.89	6.47	9.62	7.75	8.41	9.51	61.22
Royal Palm Ranger Station													
1945-52	—	—	—	—	—	—	—	—	—	—	—	—	—
1953-61	6.23	2.02	1.35	1.81	1.93	2.17	2.65	7.23	8.72	6.70	7.26	7.64	55.71
1962-67	6.01	2.17	.92	1.33	1.66	.96	1.76	4.47	13.11	6.50	6.06	11.67	56.62
1968-82	5.81	2.20	1.56	1.75	2.03	1.40	3.50	7.25	10.71	7.00	8.33	9.01	60.55
1983-89	3.16	3.32	1.46	2.28	1.87	2.63	1.60	4.83	5.76	8.19	8.66	7.49	51.25
1949-89	5.64	2.40	1.40	1.67	1.93	1.75	2.75	6.20	9.04	7.30	7.86	8.60	56.54
Dade-Broward Levee/Trail Glades Range													
1945-52	7.66	1.09	1.09	.73	1.25	1.74	3.60	2.77	5.81	7.19	5.62	9.64	48.19
1953-61	7.12	1.54	1.46	2.27	1.52	1.71	4.36	5.10	6.94	6.12	6.33	6.97	51.44
1962-67	4.94	1.07	1.69	1.82	2.70	1.55	1.41	3.13	12.53	5.15	5.57	7.72	49.28
1968-82	3.10	2.65	1.02	1.56	2.28	1.26	2.84	5.91	6.60	5.69	8.46	7.43	48.80
1983-89	3.24	3.80	1.97	3.20	1.50	4.80	1.44	4.30	6.53	6.12	8.10	5.37	50.37
1945-89	5.34	1.96	1.39	1.72	1.83	2.01	2.87	4.32	7.51	6.10	6.82	7.73	49.60

intense precipitation are missing (months with missing data are omitted from the computation of means). Amounts reported from Forty-Mile Bend and the Royal Palm Ranger Station are successively higher, and the highest reported annual totals are at the Homestead Agricultural Experiment Station. The latter station is the easternmost, located on the coastal ridge; the others are inland in the Everglades. At the Miami International Airport, also located on the coastal ridge, annual totals generally are intermediate between those from Royal Palm and Homestead.

Long-term trends generally are not evident. One exception is that October rainfall averages show a marked decrease with time at all sites. Also, at some stations, annual totals and late summer totals seem to decrease in the most recent time period (water years 1983-89). Within each year, about 75 to 80 percent of precipitation occurs between May and October. December has the least rainfall, and June and September have the most rainfall. A study of May and October rainfall revealed that, on the average, two-thirds of May rainfall occurred in the latter half of the month, and two-thirds of October rainfall occurred in the first half of the month.

In the most recent water-management time period (water years 1983-89), the existence of many more rainfall stations than provided data in earlier years

Table 9. Summer rainfall totals from 1983 to 1989 at selected measuring stations

[Where available data are insufficient, no value is computed. 1985-89 averages are indicated in figure 26. Location symbols in figure 26 are defined in table 7. **Bold** indicates a few days of data are missing; e, estimated data]

Station location	Summer (May-October) rainfall totals, in inches							Average	
	1983	1984	1985	1986	1987	1988	1989	1985-89	1985-87
—	**Northwest (Upper Shark River Slough)**								
Tamiami Trail at Forty-Mile Bend	40.00	22.97	—	—	35.05	**45.09**	—	—	—
L-29 at S-12D	—	—	**33.91**	35.38	43.44	47.35	23.71	36.76	37.58
NP-201	—	—	—	—	34.10	**39.27**	23.29	—	—
NP-203	—	36.23	45.14	33.30	**34.81**	45.14	**27.55**	37.18	37.75
Northeast Shark River Slough No. 1	—	—	—	33.44	—	**49.23**	—	—	—
	Central-West (Lower Shark River Slough/Western Rocky Glades								
NP-206	—	—	33.94	**29.58**	**44.09**	51.37	**28.97**	37.59	35.87
P-36	—	**23.06**	—	—	**27.23**	51.24	**28.13**	—	—
P-35	—	31.39	—	**32.80**	**20.41**	**40.89**	30.23	—	—
	Southwest (East of Shark River Slough)								
P-38	—	32.32	**26.78**	30.87	24.55	**51.99**	32.60	33.36	27.40
	North-Central Boundary (Tamiami Canal)								
Trail Glades Range entrance	**39.50**	—	—	37.20	45.00	—	—	—	—
S-336 spillway	35.07	35.99	**32.09**	**20.97**	34.56	40.96	**28.56**	31.42	29.21
	North-Central (Rocky Glades)								
Chekika State Recreation Area	—	—	40.62	41.58	44.01	60.47	41.28	45.59	42.07
S-331 pumping station	43.50	46.64	41.14	33.46	**42.46**	49.81	32.77	39.93	39.02
	Central-East (Homestead Area)								
Homestead Agricultural Experiment Station	43.79	44.26	33.20	32.98	37.80	62.90	—	—	34.66
Homestead Field Station	42.98	41.66	**34.43**	33.34	39.37	56.64	33.56	39.47	35.71
	Central-West (Royal Palm Ranger Station Area)								
Royal Palm Ranger Station	37.64	39.77	39.40	—	34.68	53.17	30.40	—	—
S-332 pumping station	39.12	**35.41**	35.85	**30.04**	36.23	**58.68**	27.03	37.57	34.04
	South-Central (Southern Glades)								
S-18C spillway	31.31	29.03	37.39	21.13	25.47	45.23	27.22	31.29	28.00
	Central-East Boundary (Eastern Glades)								
S-20F spillway	32.50	**23.26**	35.52	23.84	30.56	62.16	**34.11**	37.24	29.97
	Southeast Boundary (Southeastern Glades)								
S-20 spillway	21.96	27.50	**35.49**	18.98	26.84	36.47	**24.30**	28.42	27.10
	Miami International Airport								
Northeast (Miami)	33.18	43.77	46.39	41.12	33.38	39.01	32.83e	38.54e	40.30

permits a more-detailed analysis for areal variations. At 21 stations, summer (May-October) rainfall totals were computed (table 9) for each year between 1983 and 1989 for which data were available. Summer values were selected for comparison because they represented most of the annual total and were considered to be the part of annual rainfall most likely to show areal variations. Five-year averages (1985-89) computed for all stations with sufficient data are also listed in table 9 and shown in figure 26. In a few cases, 1983 or 1984 data were available, but it was found that 6- or 7-year averages were within 1 in. of the 5-year averages. One exception was S-20F for which 6- and 7-year averages were 2.5 in. less, possibly because missing-day values for 1984 would have added significant rainfall amounts.

Despite the greater amount and distribution of rainfall data from recent years, inferences concerning areal variations are still difficult to make without having additional sites. Results of the analysis indicate that summer rainfall totals are similar in the Shark River Slough, upper Taylor Slough (S-332 pumping station), and the east coast (S-20F spillway), and only slightly higher in Homestead (Homestead field station) and the eastern rocky glades (S-331 pumping station). An anomalously high summer rainfall average was computed for the station at Chekika State Recreation Area. Substantially lower summer averages were computed for the Tamiami

Trail (S-336 spillway) and throughout the southern glades (P-38, S-18C spillway, and S-20 spillway). Miami International Airport values seem to be similar to those from the main parts of the study area. An exact average could not be computed because August 1989 data were not available. However, several other stations had about 9 in. of rain in August. An average was computed by estimating this amount for Miami International Airport.

The 1985-89 averages indicate a lessening of average summer rainfall toward the southern and southeastern coasts of the mainland peninsula but fail to show spatial variations inland. However, if the statistical analysis is restricted to a 3-year period (1985-87), the resulting 3-year averages would indicate that upper Shark River Slough and rocky glades stations (S-12D, NP-203, Chekika State Recreation Area, and S-331) averaged 4 in. more rainfall each summer than the southern coastal ridge stations (the two Homestead stations and S-332). This evidence would correlate with observations of residents of the area that summer thunderstorms often develop over the Everglades while coastal ridge communities remain dry. If such a weather pattern tended to prevail in some summer wet seasons, there could be a disparity in the spatial distribution of summer rainfall in some years, with the largest amounts concentrated in the north-central part of the study area.

That higher amounts of rain fall inland during the summer is supported by the nature of the weather system in eastern Dade County. The temperature contrast between land areas heated by the sun and the cooler ocean leads to a circular thermal convection system (Eric Schwartz, South Florida Water Management District, oral commun., 1993) in which onshore movement of air masses from the ocean transports moisture inland that rises to form thunderheads. The additional effects of prevailing easterly winds cause thunderstorms to develop inland away from the coast. The variation of the velocity of the easterly winds explains the variation in the location of the heaviest rains.

Evapotranspiration

Evapotranspiration refers to the combined processes of evaporation of moisture from surface water, from the unsaturated soil zone, and from the subsurface saturated zone, and action of plants in drawing water from below land surface and releasing it through leaves and woody material (transpiration). Evapotranspiration removes a substantial amount of water from the Biscayne aquifer on an annual basis. In fact, most investigators have assumed that the quantity of water that evaporates or transpires is nearly as large as the quantity recharged by precipitation.

The rate of evapotranspiration is difficult to measure directly and generally is estimated indirectly by water-budgeting procedures, including simulation modeling. The rate of evaporation of surface water can be measured directly (pan evaporation), but the rate of evapotranspiration below ground surface is less than evaporation from the surface and is assumed to decrease with depth below land surface until an extinction depth is reached, at which no water is removed by evaporation or plant transpiration. The decrease of evapotranspiration with depth is considered to be partly a function of the depth of shallow and deep root zones, corresponding, respectively, to roots of grasses and small plants and to roots of trees and other large, deep-rooted plants. Application of this concept to southern Dade County is ambiguous because, in much of the area, less than 1 ft of soil overlies hard limestone rock that might not be penetrated by some tree roots. However, the porosity of the limestone may permit evaporation to occur by other means.

Some investigators consider the rate of evapotranspiration to vary according to the type of land use or vegetation cover characteristic of an area, and ongoing research in Dade County seeks to quantify such relations. Studies in southern Florida (Stephens and Stewart, 1963) have shown that the degree of evapotranspiration is closely correlated with the amount of solar radiation, which implies a strongly seasonal variation (high in summer and low in winter). An experimental study in Broward County (Stewart and Mills, 1967) showed this effect and also a significant relation to density of plant cover. The depth to water table ranged from 1 to 3 ft in outdoor laboratory tests, but this had a significant effect on the evapotranspiration rate only during periods of low rainfall.

Long-term hydrographs of water levels in wells in Dade County (fig. 7) do not seem to indicate a relation between depth below land surface and the evapotranspiration rate. At wells G-1362 and S-196A, dry-season water levels are 10 to 11 ft below land surface (fig. 7), but the recession curves show no change of slope that would indicate a reduction in the rate of evapotranspiration. Such an effect, however, might be obscured by delayed aquifer drainage, increased plant growth, and higher solar radiation during the late spring months.

SIMULATION OF THE WATER-TABLE ALTITUDE IN THE BISCAYNE AQUIFER IN SOUTHERN DADE COUNTY

The examination of selected data describing the hydrologic cycle of southern Dade County has provided the basis for the construction of a numerical model of flow in the Biscayne aquifer. The model construction also requires that innovative approaches be devised to resolve several significant representational problems. Once the numerical model is constructed and calibrated, it will be used for the analysis of selected aspects of the system of ground water and surface water in the region. This will help to elucidate the behavior of the model and to identify the strengths and weaknesses of the methods used to represent the aquifer flow system.

Approach

The simulation of the flow regime in the surficial Biscayne aquifer of southeastern Florida is a problem with many aspects that constrain the straightforward application of standard modeling techniques. Nevertheless, because the aquifer is the sole source of drinking water for millions of people and is especially vulnerable to contamination, many attempts have been made to model aquifer flows. When the geographical area represented in model grids has been small, many of the typical simulation problems have been avoided. A few simulations have been attempted on a regional scale with reasonable success, subject to qualifications inherent in the simulation methods used (see “Previous Studies”). Because of the difficulty in obtaining a scientifically rigorous solution, USGS involvement in surficial aquifer system flow simulation in southern Florida has been minimal in recent years since construction of the analog model by Appel (1973).

The simulation described herein attempts to represent flows in a large region of the Biscayne aquifer lying south of Tamiami Trail in Dade County. The objective was to devise and apply a simulation strategy that deals with the special problems of regional simulation in this area in an innovative and scientifically rigorous manner. This objective was to be accomplished in an efficient and economical way by minimal modification of an existing simulation code and by innovative application techniques. A description of the code selected for the task and a description of how simulation problems were addressed are presented in the following sections.

Simulation Code

The code selected for the simulation was the Subsurface Waste Injection Program (SWIP). This selection was made for three reasons: (1) the author’s close familiarity with details of the code, which permitted many complex modifications to be made with relative ease and confidence; (2) the need for a solute-transport code for subsequent analysis of a chloride plume in the area of simulation; and (3) the author’s perception that the computational methods of the code lent themselves in an advantageous way to desired modifications.

The SWIP code was developed by INTERCOMP Resource Development and Engineering, Inc. (1976) under sponsorship of the USGS. The code was later revised for the USGS by the same firm renamed INTERA Environmental Consultants, Inc. (1979). Despite its intended use as a special package for waste-injection problems, the code was treated within the USGS as a general-purpose three-dimensional simulator of solute and thermal-energy transport in ground water (Robson and Saulnier, 1981; Merritt, 1986; and Hickey, 1989). Outside the agency, the SWIP code has been adapted for special purposes by various public and private organizations (Battelle Memorial Institute, Sandia National Laboratories). The SWIP code has been used as a resource in USGS efforts to develop newer three-dimensional solute-transport simulators with enhanced capabilities (Kipp, 1987).

Absolute pressure is the solution variable of the flow equation, and the model accounts for fluid density and viscosity dependence on temporal changes of pressure, temperature, and solute concentration. The solution variables are expressed in residual notation (for example, $p = p_o + \delta p$, where p_o is assigned a value from the previous timestep), and the equations are solved for values of the residuals (δp). This has the advantage of reducing roundoff error. The solution of equations for pressure, solute concentration, and temperature is by standard finite-difference techniques. The aquifer simulated may be fully confined or have a free upper surface, and the equations may be solved in Cartesian or cylindrical coordinates. In every problem, a set of values is initially assigned to the pressure, temperature, and concentration fields, and specifications are required for all hydraulic, solute, and thermal transport parameters. However, if only one or two of the solution variables are to be determined, only the equations for those variables are solved. Boundary conditions for pressure, temperature, and concentration must be specified at all active boundary faces. In this study, only the flow equation was solved; equations for solute and thermal transport were not solved and assigned initial values remained unchanged.

The free-surface simulation capability was a routine made available in the 1979 version of the code upon the request of the USGS, and some minor errors required correction by the author for it to work properly. In the SWIP code, the pressure variable can take any numerical value. When free-surface calculations are performed, pressures at the nodal centers of grid cells in the uppermost layer are compared with a user-specified base pressure to determine the saturated thickness of the cells. Internodal conductances are then appropriately modified, as are mass-balance terms. In the uppermost layer of free-surface cells, a special algorithm for computing changes in storage is used in which effective porosity (approximately equal to the specific yield) is the dominant term. In lower layers that are fully saturated, the change in storage term is computed as an algebraic combination of effective porosity and compressibility values (Lohman, 1979, p. 8) as is standard practice in simulating confined aquifer conditions.

For this study, upper layer cells were allowed to become dry and an approximation was developed to represent the subsequent rewetting of the dry cells (Merritt, 1994). Because the presence of dry cells in the upper layer implied partial saturation and the need for modification of internodal conductances in lower layers, the free surface test against a base pressure value was generalized and extended to cells in all layers of the model grid.

The author modified the SWIP code to compute an evapotranspiration rate at all nodes designated as recharge nodes (INTERA Environmental Consultants, Inc., 1979). A maximum evapotranspiration rate and two depths, those of the bottom of shallow and deep root zones, are specified. Evapotranspiration is considered to occur at the maximum rate above the bottom of the shallow-root zone and to decrease linearly from below the bottom of the shallow-root zone to the bottom of the deep-root zone, which is, therefore, an extinction depth. The function was coded implicitly, as recommended by Trescott and others (1976, p. 8), to ensure numerical stability.

In this application of the SWIP code, boundary conditions for pressure, temperature, and concentration are specified at the centers of the outer faces of boundary cells on all lateral and vertical surfaces of the grid. Boundary fluxes are computed based on the difference between the boundary pressures and those computed at the boundary grid-cell centers. If a boundary specification is not made at some boundary cell face, no flux across the boundary face is computed; that is, the boundary face is treated as a no-flow boundary. For this study, the model was given the capability to allow boundary pressure values to vary with time according to user specification. The specified variation may either be cyclic with annual periodicity or may be continuous (noncyclical).

Special Simulation Problems

Before the model of flows in the Biscayne aquifer of southern Dade County could be constructed, certain problems in developing the simulation approach required consideration. The temporal scale and definition best suited for the solution had to be selected. Representational methods were needed for the simulation of overland sheetflow over a land surface of spatially variable altitude, for surface flows in canals hydraulically connected to the aquifer and the restriction of canal flows by control structures, and for the impoundment of overland sheetflow by levees. A rewetting approximation was needed so that parts of the overland sheetflow layer could become partially saturated again after seasonal drying. A method of specifying boundary conditions realistically at locations where generic, time-invariant natural boundaries were not present needed development and had to be consistent with the temporal scale and definition of the simulation. The resolution of these problems is described in the following sections.

Transient or Steady-State Simulations

Generally, flow models can be calibrated by comparing computed hydraulic heads with a naturally time-invariant equilibrium set of head measurements (steady-state calibration) or with time-varying head measurements (transient calibration). The steady-state approach has the advantage that lengthy computations are avoided. The storativity of the aquifer is not a factor in the solution, which eliminates data requirements to support estimates of parameters related to it (storage coefficient, porosity, and rock compressibility). Transient simulations have the advantage that the time-varying hydraulic behavior of the aquifer can be simulated, increasing the degree of confidence that hydraulic processes have been adequately represented.

Natural time-invariant conditions are uncommon in some aquifers. This is particularly true in surficial aquifers, such as the Biscayne aquifer, which are strongly influenced by seasonally varying rainfall and evapotranspiration rates. Nevertheless, some studies in Florida have attempted simulations of average dry-season and average wet-season conditions or of average annual conditions. The first type of simulation might not be consistent with the steady-state assumption because surficial aquifer heads during the dry season in southern Florida are rarely in true equilibrium. Summer wet-season heads can approximate an equilibrium rate after the initial buildup, though the equilibrium assumption still needs to be supported by a review of various types of data. Steady-state simulations of these types can produce useful interpretations, but results should be appropriately qualified. Time-average solutions that do not assume steady-state are also possible (Prych, 1983). Average annual conditions can be somewhat abstract. Even though a steady-state simulation of average annual conditions may be theoretically valid, important facets of aquifer behavior, such as the response to natural or manmade stresses of a seasonal periodicity, are ignored.

In southern Florida, the response of the water table to seasonal changes in the rates of recharge or depletion of aquifer storage is an important facet of surficial aquifer behavior. Because a major objective of this study was to achieve the most realistic possible simulation, transient simulations were the sole focus of the study. Nevertheless, attempts were first made to reduce the computational effort by doing transient simulations in an “average” sense, in which boundary conditions, rainfall, and comparison head data were averaged per month over each of the five water-management time periods previously described (figs. 16, 17, and 19-22). This approach led to confusing results, however, as rainfall data errors in some months made rainfall averages inaccurate, with the result that precise simulation of “average heads” was unachievable with calibration parameters that worked in all water-management time periods.

When real-time transient simulations were attempted, rainfall data errors and other specification errors were more readily identified. Despite the increased computational effort required, it became easier to achieve water-management time-period simulations that were based on a global set of calibration parameters. The result, the actual simulation of stage and water-level data at most measurement locations in southern Dade County in every water year from 1945 to 1989, was the most ambitious goal that could have been sought from the study and supported many interpretive objectives.

Overland Sheetflow

The Biscayne aquifer was previously shown to be partly overlain by a seasonal body of surface water (fig. 8) that may become virtually nonexistent during the dry season (fig. 9). The surface-water stage, varying seasonally in both elevation and its areal extent, strongly influences the ground-water flow system. Thus, a rigorous approach to the simulation of aquifer flow requires the representation of the surface-water body as a control on water-table altitude and as a source or sink of water flux.

The ground-water flow system also influences the stage and areal extent of surface water. However, this influence would seem to be less significant than that of surface water on the aquifer because the storage coefficient of the aquifer is much less than the specific yield of surface water. Therefore, a greater volume of water must be added or removed to change the surface-water stage than is required to change the water-table altitude by a similar amount. The volume of ground water available to be released to surface-water bodies is limited both by the lower storage factor and by the higher resistance to flow within the aquifer.

The interaction of the Biscayne aquifer and flow in the Everglades is limited by their partial geographic separation. As shown in figure 3, the lower part of Shark River Slough lies to the west of the western limit of the Biscayne aquifer established by Fish and Stewart (1991). However, surface waters in Northeast Shark River Slough and in the southern glades directly overlie highly permeable sections of the Biscayne aquifer, separated only by thin layers of peat or calcitic mud. Even that slight confining layer is not present where surface waters periodically flood parts of the rocky glades.

Because the extent of the overland flow region varied substantially during the year, the manipulation of boundary conditions to represent the influence of the sheetflow region was not considered to be a practical solution. Instead, the surface-flow system was incorporated into the model grid as an upper layer. Inclusion of the surface-water features in the model grid required either that the Darcian flow approximation be used to compute surface-water flows, with "equivalent hydraulic conductivity" coefficients used to represent the resistance to flow, or that the simulator be modified to provide an alternative computational algorithm for surface-water flows.

To compare these alternatives and to investigate the relation between surface-water and ground-water flow velocity estimators, the Manning equation, which relates the rate of surface-water flow to the energy gradient, was compared with the Darcian approximation for ground-water flow. According to Chow (1959), the Manning equation, an empirically derived formula found to be reasonably accurate for a wide variety of field conditions, "has become the most widely used of all uniform-flow formulas for open-channel flow computations." Chow (1964) presents a form of the equation modified for overland flow:

$$v = \frac{1.486}{n} D_a^{0.67} S^{0.50} \qquad (1)$$

where:

v is velocity, in feet per second;

n is a coefficient of roughness;

D_a is the average flow depth, in feet; and

S is the energy slope, approximately equal to the absolute value of dh/dl, the slope in hydraulic head, in feet per foot, where 1 is a length coordinate in the direction of flow.

Recalling that SWIP uses a residual formulation for the solution variables (including pressure) to reduce roundoff error, and assuming that ground-water flow is confined to a horizontal plane and occurs in a direction with length coordinate 1, its velocity is estimated by Darcy's equation as:

$$u = -K\frac{dh}{dl} = -\frac{144\ K}{\rho}\frac{dp}{dl} = -\frac{144\ K}{\rho}\left(\frac{dp_o}{dl} + \frac{d\delta p}{dl}\right) = u_o + \delta u \qquad (2)$$

where:

K is hydraulic conductivity, in feet per day;

h is hydraulic head, in feet;

u is Darcian velocity, in feet per day;

ρ is weight density, in pounds per cubic foot.

p is pressure, in pounds per square inch;

p_o is the value of p at time t_o;

δp is the incremental correction to p_o computed by SWIP at the current $t = t_o + \delta t$;

u_o is the Darcian velocity at time t_o; and

δu is the incremental correction to u_o computed by SWIP at the current time resulting from the incremental change in the pressure gradient.

If a form of the Manning equation (eq. 1) is to be used in SWIP, in which the solution matrix is linear in dh/dl, it also must be expressed as a linear function of $(\delta p/\delta l)$. This is done by writing the head gradient as a function of the incremental pressure gradient as in equation 2 and applying a Taylor series approximation:

$$v = \frac{1.486}{n} D_a^{0.67} \frac{12}{\rho^{0.50}}\left(\frac{dp_o}{dl} + \frac{d\delta p}{dl}\right)^{1/2} \qquad (3)$$

$$\cong \frac{1.486}{n} D_a^{0.67} \frac{12}{\rho^{0.50}}\left[\left(\frac{dp_o}{dl}\right)^{0.50} + \frac{1}{2\left(\frac{dp_o}{dl}\right)^{0.50}}\frac{d\delta p}{dl}\right] = v_o + \delta v$$

where:

v_o is the fluid velocity at time t_o, and

δv is an incremental correction computed at the current time resulting from the incremental change in the pressure gradient.

The approximation is reasonably accurate when:

$$\frac{d\delta p}{dl} \ll \frac{dp_o}{dl} \qquad (4)$$

This criterion will be satisfied when pressure changes are relatively uniform spatially compared to the pressure gradient, which may be true in stable computations at a sufficiently small time increment when stages rise or fall evenly across the flooded area and the sheetflow retains a finite slope.

An attempt to integrate a linearized approximation of the Manning equation directly into the solution scheme of the SWIP code was not successful, possibly because of coding errors. Rather than pursue this alternative, it was decided to take a more direct approach by interpreting K in the Darcian flow equation as the equivalent hydraulic conductivity representing the resistance to flow in the overland flow layer. For K in equation 2 to represent the proportionality factor of the linearized Manning equation requires that $-\delta u = \delta v$, or:

$$86,400 \bullet \frac{1.486}{n} D_a^{0.67} \frac{1}{2\left(\frac{dp_o}{dl}\right)^{0.50}} \bullet \frac{12}{\rho^{0.50}} = \frac{144}{\rho} K' \tag{5}$$

or:

$$K' = 5349.6 \frac{D_a^{0.67} \rho^{0.50}}{n\left(\frac{dp_o}{dl}\right)^{0.50}} \tag{6}$$

where K' denotes the equivalent hydraulic conductivity of the surface-water body and the constant 86,400 converts feet per second to feet per day.

Using the estimate for K' of equation 6, the approximation to the Manning equation for flow now has the same form as that for Darcian flow. It should be noted that the derivation of this formulation depended upon the use of the residual notation in the SWIP solution. K' is a function of the head gradient and depth of flow and is, therefore, not constant in time or space. In the sheetflow region of the Everglades, neither the head gradient nor the depth of flow varies greatly with location. The head gradient tends to vary little in time as well. However, the depth of flow varies seasonally from 0 to 2.5 ft. The dependence on the depth of flow cannot be replicated when a constant value is assigned to the hydraulic conductivity coefficients for overland flow in the SWIP model grid.

The foregoing discussion illustrates the limitations inherent in using the hydraulic conductivity value assigned by the SWIP code to the surface layer of overland flow as a calibration parameter that approximates the Manning equation computation. However, recognizing that the Manning equation is empirical and justified by its accuracy in replicating measurements, the representation of overland flow by the use of equivalent hydraulic conductivity coefficients that were invariant in space and time was accepted for use in this study as an empirical approach to be evaluated on the basis of its accuracy in replicating observed data.

It should be noted that the appropriateness of the Manning equation (developed for turbulent flow regimes) in representing overland flow, which may be nearly laminar because of its low velocity and the high resistance encountered from vegetation and irregular surfaces, has been questioned by some investigators (Maheshwari, 1992). Maheshwari notes that overland flow is highly subdivided, as is flow through porous media, which implies that the dependence on the energy slope may be nearly linear.

It is of interest to estimate the equivalent hydraulic conductivity values that might apply to overland flow in the study area. A representative value of K at a hypothetical point in time and space may be estimated by assuming that the roughness coefficient, n, is 0.05; the depth of flow, D_a, is 1 ft; water density, ρ, is 62.4 lb/ft^3 (pounds per cubic foot); and the pressure gradient in the overland region of the Everglades, dp_o/dl, is 4 ft (1.7 lb/in^2 [pounds per square inch]) in 15 mi (a typical wet-season gradient). With these idealized assumptions, $K' \cong 1.8 \times 10^8$ ft/d. This demonstrates the approximate value of K' that might be needed to calibrate the model to represent stages in the sheetflow region of the Everglades.

The approximations required in the derivation of the equivalent hydraulic conductivity (K') indicate the possibility that an accurate calibration of both stage and flow rate might not be achievable by these means. Therefore, the more limited objective of the calibration process was to obtain a generally accurate simulation of surface-water stages. If this were accomplished, the influence of the surface-water body on ground-water flow would be accurately represented, achieving the principal objective of the model development.

Rainfall recharge was applied to the uppermost partially saturated cells in each vertical column. For compatibility with the treatment of overland flow, the maximum evapotranspiration rate was applied to the overland flow layer grid cell when it was wet, but the evapotranspiration rate was reduced according to depth to water below land surface and applied to the uppermost partially saturated grid cell when the overland flow layer grid cell was dry. Land surface was considered to be the boundary between the first and second layers for this computation.

Rewetting

In the 1979 version of the SWIP code, it is assumed that the water surface does not drop below the bottom of the surface layer of grid cells. When this situation does occur, it is treated as an error condition. However, in many parts of Dade County that are never covered by surface flow, the corresponding grid cells in a model that includes a surface-flow layer would be permanently dry, and a free surface would exist below the top of the second or some lower layer of cells. In parts of the county where surface flows occur seasonally, parts of the surface-flow layer would periodically vary between being dry and being partly saturated, and underlying layers would vary between being partly or fully saturated.

The code was, therefore, modified so that grid cells in all layers of the model were tested to determine if a free surface occurred within them (if the pressure at the top of the cell were less than the specified base pressure). A more complex problem was enabling the code to "rewet" a cell that had become dry (when the pressure at the center of the bottom face of the grid cell was less than the specified base pressure). The 1979 version of SWIP assigns a zero value to internodal conductance when one cell is dry, which effectively prohibits water from ever again entering the grid cell from adjacent grid cells after it has become dry. (The internodal conductance, based on the hydraulic conductivities of adjacent grid cells, defines the flow of water between the grid cells for a specified head gradient.)

The rewetting approximation developed for this study is described separately by the author (Merritt, 1994). Dry-cell pressures are not normally updated in successive computational timesteps because of the absence of fluxes into or from the dry cells. The SWIP code was modified to adjust dry-cell pressures after each timestep by the amount computed for the uppermost nondry cell in the vertical column. Eventually, as simulated recharge exceeded simulated discharge from the lower cells, the dry cells were assigned pressures that made them nondry and were treated as normal nondry cells.

Land-Surface Definition and Levees

Because the depth of overland flow is rarely greater than 2.0 ft, overland flow was entirely contained within the thickness of the surface layer of partially saturated overland flow grid cells. The top of the second layer of grid cells was assumed to represent land surface. The relation between the upper two layers is illustrated schematically in figure 27. This procedure required a detailed description of land surface in the model so that the drying and rewetting of the overland flow layer could be accurately represented.

The SWIP options to vary cell elevations and thickness within layers were used to vary the elevation of the upper surface of the second layer of grid cells. Various local and regional land-surface elevation maps and control elevations on USGS 1:24,000 topographic maps were used for guidance. The detail required to accomplish this task and other grid modifications proved cumbersome; between 1,000 and 1,500 instruction lines in the input data file were required.

For convenience, the bottom of the second layer was set uniformly at sea level; thus, the thickness of the second layer was equivalent to land-surface elevation throughout the study area. Assigned thicknesses varied between 0.5 and 12 ft. Generally, the maximum assigned land-surface elevation values were 10 ft. These and higher elevations occur in the southern coastal ridge, where land-surface elevation is quite irregular and variations of several feet occur over small distances. Because this area virtually always remained dry, it was not considered necessary to accurately represent the higher elevations that occurred in small, isolated sections of the southern coastal ridge. Figure 28 shows the land-surface elevation variation encoded in the model runs. Each intersection of lines corresponds to the nodal center of a model grid cell. The northeast corner is highest in elevation; this represents the coastal ridge, which extends southwestward into the high ground of Everglades National Park. From the center of the northern boundary, trending southwestward, is a low area that represents Shark River Slough. North-south channels in the center of the grid are a generalized representation of lower areas in the rocky glades that channel episodic surface flows southward to Taylor Slough. The coarse grid definition of the regional flow model did not permit a highly detailed definition of these features. Strings of low-elevation nodes transecting the coastal ridge and representing the transverse glades (or canals in later time periods) could not be shown at the scale of figure 28. The southernmost row was elevated 0.3 ft above adjacent rows to the north to generally represent the effect of the Buttonwood Embankment.

In some parts of the modeled region, overland flow is blocked by levees that cause abrupt stage discontinuities. Levees were represented in the model as close to their geographic location as feasible using strings of 150-ft wide grid cells. Low hydraulic conductivity values were assigned to these cells in the upper (overland flow) layer. Where overland flow occurred, this specification served to separate surface-water stages laterally so that the effect of the levees was correctly simulated.

Canals and Control Structures

The numerous canals excavated in southern Dade County (table 3) as part of the development of the water-management system have had major effects on the flow regime in the Biscayne aquifer, particularly when control structures are opened to release

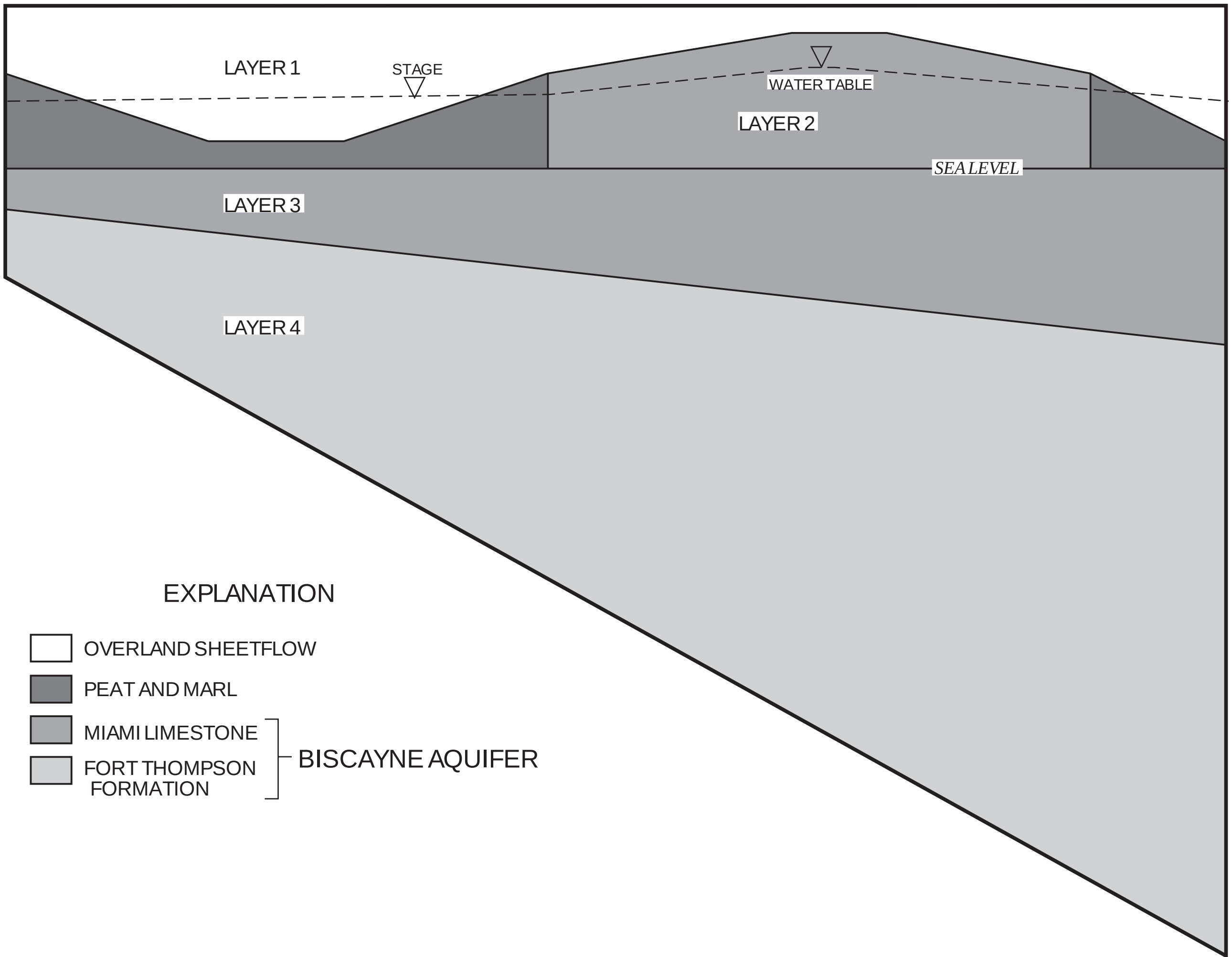


Figure 27. The vertical discretization of the model of flows in the Biscayne aquifer in southern Dade County.

floodwaters from the interior or when coastal controls are closed to maintain a water-table altitude higher than sea level. This fact requires that the hydraulic influence of the canals receive representation in a regional ground-water flow model. When calibrated, the model may be used to assess the probable effect of the canals on the ground-water flow system.

The 1979 version of the SWIP code does not have a river simulator, though a river package was developed by K.L. Kipp (U.S. Geological Survey, written commun., 1981). Most traditional flow simulators treat rivers (canals) as constant-head line sources or sinks that contribute water to or receive water from the aquifer through a leaky confining layer. Inherent in this treatment is the assumption that the river or canal stage is unaffected by water-table fluctuations in the aquifer, which in turn, implies that: (1) the confining layers are effective in separating river (canal) and aquifer heads so that the fluxes across the layer occur at a rate too slow for the river (canal) stage to be affected by changes in water-table altitude, or (2) the canal cross section is recharged at such a rapid rate that the stage does not change even if large fluxes between the river (canal) and aquifer occur.

However, it was previously shown that canal stages fluctuate readily in response to changes in ground-water heads (figs. 23 and 24), indicating that fluxes between the canals and aquifer occur with sufficient rapidity that the heads are normally equal. Only when rapid canal-stage fluctuations occur as a result of surface-water routing are stage separation effects apparent. Even in this case, the two heads vary together with a nearly constant separation that provides the impetus for the interchange of the large volumes of water required to maintain the close relation of heads.

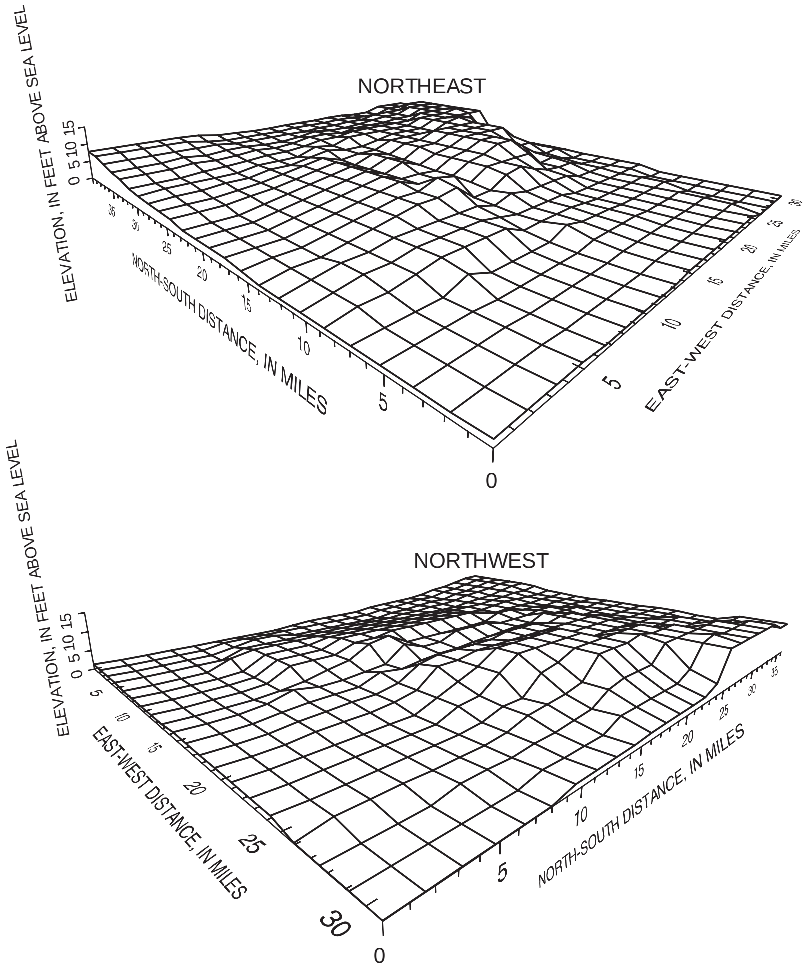


Figure 28. Three-dimensional representation of land surface in the modeled region.

The method developed for representing the influence of the canals was to incorporate them within the model domain, a procedure analogous to the treatment of overland flow. Thin strings of grid cells were incorporated as rows and columns into the planar grid design. Canals were identified with the strings of grid cells in layer 2, which generally represented the interval between land surface and sea level. However, the canal cells were deepened to have somewhat greater and more representative thicknesses. By analogy with the treatment of overland flow, equivalent hydraulic conductivities were assigned to the strings of cells in layer 2 that represented canals. Because canal flow depths may exceed 10 ft and are subject to less extreme resistance than overland flow in the Everglades, values were expected to be higher than those for overland flow.

To estimate the equivalent hydraulic conductivity values that might represent the rate of canal flows, representative values of various parameters for a hypothetical canal reach were used for a computation of equivalent hydraulic conductivity using equation 6. Because canals in the study area are unlined, excavated into limerock with a rough surface, and are often choked with aquatic weeds, a roughness coefficient of 0.05 was assumed. Observing that a stage difference between S-338 and S-148 in Black Creek Canal (C-1) of about 2.2 ft (0.95 lb/in^2) occurs in about 10 mi when the two structures were open in August 1984 (fig. 24), and substituting a hydraulic radius of 8.33 ft (100 ft wide and 10 ft deep) for the depth of flow (D_a), the equivalent hydraulic conductivity estimated by equation would be about 8.3 x 10^8 ft/d. Because most canals in Dade County are in areas rarely subject to flooding and all are deep enough so that they have never become dry since they were constructed, the canal grid cells were always partly saturated and overlain by dry overland flow grid cells in model computations.

Although southern Dade County canals were known to have nearly impervious bottom layers of sediments, the thin rows and columns of canal grid cells were treated in the model as being in vertical and horizontal communication with the aquifer surrounding and underlying them. Because the permeable canal walls penetrate depths equivalent to layer 3 of the model, no error was introduced into the regional-scale flow simulation by this generalization. To have explicitly simulated the bottom layers would have required another model layer and would not have altered the hydraulic influence of the canals on regional ground-water flow. In a model of a much smaller area, the influence of a bottom confining layer might require explicit representation.

The approach to representing canals precluded the representation of any confining layers separating canal and aquifer heads, so that the principal retarding influence on exchange of flow was the resistance to flow within the aquifer. Although this is generally consistent with previously cited data showing a close correlation between canal stages and local water-table altitudes, the approach does not account for any plugging that may reduce hydraulic interconnection in the lower reaches of some canals. Subject to this reservation, the assumption of direct hydraulic connection inherent in the selected approach is accepted as an adequate approximation for a regional-scale model of ground-water flow in the study area.

Most canals were represented as being 100 ft wide and extending about 15 ft below land surface. Smaller dimensions were assigned to cell strings representing the Grossman Road borrow ditch, and shallower depths were assigned to the North and Florida City Canals. For economy, grid cells adjacent to the canal strings typically had lateral dimensions 10 to 50 times greater than the assigned canal widths. As a consequence, the interrelation of canal stages with nearby aquifer heads was not simulated in detail. However, the regional-scale influence of canals as “short circuits” in the upper part of the aquifer was adequately represented. Because of the relatively small canal cell widths and because a cell width-weighted harmonic mean formula was used to compute the internodal conductance between canal and aquifer cells, the internodal conductance was representative of the aquifer hydraulic conductivity rather than of the canal equivalent hydraulic conductivity.

An additional difficulty in simulating canals in the study area is posed by the existence of control structures (table 4) that, when closed, separate stages in the canals. To simulate control-structure operation at the appropriate locations, algorithms were added to the SWIP code that compared upstream canal stages (the head in an upstream canal grid cell) with a control elevation. When the upstream head was found to be less than the control elevation, the internodal transmissibility was set equal to zero to prevent flow in the canal cell string, simulating closure of the structure. The specified control elevations were allowed to vary monthly.

The control-structure algorithm was made more realistic by coding the algorithm to recognize five successive stages within 0.5 ft of the control elevation, within which successively larger or smaller partial openings would be simulated by adjusting the internodal transmissivity by successively larger or smaller correction factors. This is analogous to the actual operation, in which the automatically controlled gates are set to close at an elevation that is a fraction of a foot lower than the elevation at which they are set to open and may temporarily be partly open at intermediate elevations. This procedure has advantages for computational stability in that the large change in water flux in a highly transmissive canal cell string is dampened and staged gradually over several successive timesteps. Despite this procedure, however, oscillatory behavior did occur at some structure nodes at certain simulation times. Although this behavior is considered unrealistic in the construction of the flow model, C.A. Appel (U.S. Geological Survey, written commun., 1992) reports that such behavior has actually occurred at some control structures as a result of the operational rules used at certain times early in the evolution of water-management techniques.

Boundary Conditions

In many typical flow-simulation problems, the extent of the geographical area to be represented within the modeled domain is selected so that grid boundaries correspond to various natural boundaries of the aquifer where hydraulic conditions can be described in a generic manner. Examples are aquifer outcrops or pinchouts, faults, rivers with known time-invariant stages, or undersea subcrops where saltwater hydrostatic heads can be defined.

In this study, however, it was not feasible to select a geographic area entirely bounded by natural features convenient for modeling purposes. Although the Biscayne aquifer thins and becomes less permeable in the northwestern part of the study area, the aquifer extends far beyond the study area to the north, east, south, and southwest. In the latter three directions, little information exists to describe characteristics of the aquifer where it is saltwater intruded, although subcrops on the edge of the Atlantic Shelf are probable. The search for natural aquifer boundaries, therefore, would have required inclusion of many counties to the north of the study area, where hydrologic and water-management features are as complex as those in the study area, and extension of the modeled area to the eastern, southern, and southwestern edges of the Atlantic Shelf.

An alternative to the use of Atlantic Shelf subcrops as eastern and southern model boundaries was to put model boundaries at locations corresponding to the coasts of Biscayne Bay and Florida Bay, where aquifer heads are controlled by accurately measured tidal stages that are somewhat higher than sea level and vary both seasonally and with location along the bay coasts (figs. 12 and 13). This procedure was consistent with the assumption made previously that heads in the aquifer were nearly the same as in overlying surface-water bodies. However, the existing code did not provide a means for specifying time-varying values for boundary pressures. A method was needed to convert time-varying tidal stages to boundary-pressure specifications that would be correct at each successive simulation timestep. If a convenient algorithm could be developed, the same procedure could be applied to arbitrary locations inland where sufficient water-level or stage data existed to define time-varying hydraulic conditions in the aquifer. Use of measured time-varying water levels and stages as a basis for boundary-pressure specifications would contribute to the accuracy of the transient simulations that were the objective of the study.

A subroutine was coded for inclusion with the SWIP package that processes water-level or stage history data in the form of monthly averages for conversion into pressure specifications at designated lateral boundary-cell faces. The monthly average stages or water levels were obtained from a maximum of 10 field locations (canal stage recorders or well recorders) scattered along each lateral boundary. The specified monthly averages were assumed to apply to the middle of the month. In simulations, the current computational time was identified with a particular month and day, and interpolation in time was performed between the appropriate pair of mid-month values at each field location. The resulting set of up to 10 current head values was interpolated laterally to the centers of boundary faces. The heads were then converted to hydrostatic pressures at each vertical boundary-face center on the basis of prespecified fluid densities assigned to the boundary faces. Thus, coastline pressure assignments increased vertically downward as a function of saltwater density, whereas inland pressure assignments increased vertically downward as a function of freshwater density. The entire procedure was repeated at each computational timestep.

The monthly average input routine was also used to enter other parameters that varied seasonally, including: (1) rainfall rates, (2) maximum evapotranspiration rates, (3) well pumping or injection rates, (4) structure control elevations, and (5) discharge rates at coastal control structures. Rainfall rates and structure control elevations were considered constant during each month; other values were interpolated in time. Rainfall rates for the first half of May were two-thirds the monthly average rate and were four-thirds the monthly average rate during the second half of the month. A reverse procedure was applied to rainfall rates in the first and second halves of October.

The lateral discretization of the modeled region was into 32 columns and 36 rows. Superposition of the horizontal grid on a map of the study area (fig. 29) shows locations of grid boundaries where seasonally varying pressure conditions were specified. Some row or column dimensions are so small that the rows or columns cannot be shown in the illustration; these are the rows or columns used to represent canals and levees in the later time periods.

The lower two-thirds of the eastern boundary of the grid corresponds to the coast of Biscayne Bay, Card Sound, and Barnes Sound where saltwater hydrostatic boundary pressures were specified. Stage data from tidal recorders (Biscayne Bay near Homestead and Card Sound at Model Land Canals) were used to determine the value. Tidal stages along or near the southern coast, the northern edge of Florida Bay (Florida Bay at Flamingo and Card Sound at Model Land Canals), are used to generate values for the southern boundary of the model grid, which is generalized as a straight line.

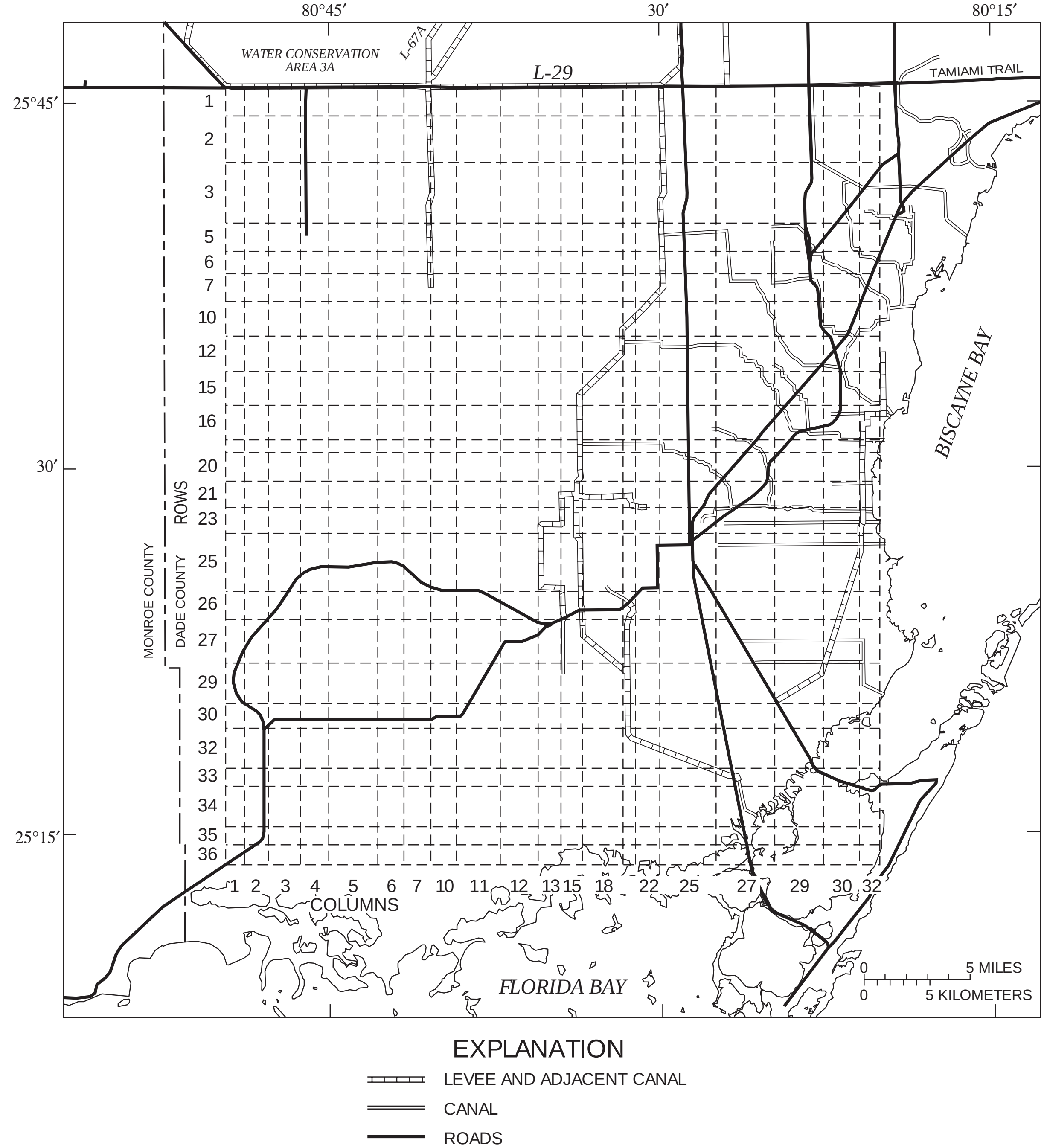


Figure 29. Study area with model grid superimposed.

The upper part of the eastern boundary of the grid was a freshwater hydrostatic boundary based on data from well and surface-water stage recorders north of the mouth of C-1 ((G-10, G-39, Tamiami Canal at Coral Gables, G-595A, G-553, and G-860). Station locations are shown in figures 16, 17, 19, 20, and 21. Pumping in the Southwest, Snapper Creek, and Alexander Orr Well Fields after 1950 was implicitly represented in the model by the boundary heads from well G-595A, located in the Alexander Orr Well Field where the greatest and most extensive drawdowns occur. The Alexander Orr Well Field is located directly on the model boundary. The Southwest Well Field is nearly 4 mi inside the boundary, but drawdowns greater than 0.5 ft rarely extend farther than 1 mi from the well field. The Snapper Creek Well

Field, which is less than 2 mi from the boundary and which began pumping in 1978, affects the water table in an even smaller area. The nearest well providing water levels for calibration was G-858, 3 mi southwest of the Southwest Well Field and more than 5 mi from the eastern boundary of the model. For these reasons and because the area of primary interest in the calibration was southwest of well G-858, the implicit representation of well-field pumping was considered an adequate approximation for purposes of this study, and well-field pumping was not explicitly represented in the model.

Long-term monitoring of stages in the Tamiami Canal (at Forty-Mile Bend, S-12C, Bridge 45, Krome Avenue, east and west of L-30, S-334, S-336, Dade-Broward Levee, and at Coral Gables and of water levels in nearby wells (G-618 and G-619) provides a substantial data base to describe the water-table altitude along this nearly straight east-west inland feature, making it convenient for use as a northern boundary. Station locations are shown in figures 16, 17, 19, 20, and 21. Stations in this group providing data after the construction of L-29 in 1962 were located south of L-29. Therefore, in designing the regional flow model, it was possible to avoid consideration of the complex history of changes in the management of Everglades surface-water flows at the Tamiami Trail by using stage data from these stations. Important changes in the flow regime in the study area that result from water-management changes at the canal boundary are implicitly represented by use of stage data and well water levels south of the levee. Inherent in this procedure is the identification of canal stages with the water-table altitude, shown earlier (fig. 23) to be a reasonably accurate and suitable assumption for construction of a regional-scale model.

Heads along the western boundary were approximated using data from Everglades surface-water stages with long periods of record (P-33, P-34, P-35, and P-38) and stage records from Tamiami Canal at Forty-Mile Bend, Tamiami Canal below S-12C, and average tidal stages from Florida Bay at Flamingo. Use of data from surface-water stations in the Everglades to describe the water-table altitude was considered a suitable approximation on a regional scale, as surface-water stages and water levels in wells were shown to correlate well (figs. 14 and 15). The western boundary was aligned with the Forty-Mile Bend station. Except for P-38, the widely scattered surface-water stations were not ideally located to describe head variations on the boundary. Stages were interpolated between P-33 and P-34 to obtain boundary values, and P-35 stages were interpolated to the boundary longitude. After 1962, the Forty-Mile Bend station was at the lower end of a closed canal segment (fig. 18), and stages might have appreciably exceeded the local water-table altitude (fig. 24). Therefore, the stage at S-12C in the unimpeded downstream reach was considered more representative of the water table in the Forty-Mile Bend area.

Of the surface-water and ground-water stations used to construct a long-term history of boundary conditions, not all have records that extend throughout the entire time period of the model (water years 1945-89). In the Everglades, monthly values before 1953 were set equal to long-term monthly averages. Along the coasts, both early and recent data are unavailable from the tidal stations (table 2), and long-term monthly averages were also used. It should be noted that the long-term average values were for sites along the downstream tidal boundaries of the model where actual stages did not vary significantly from the long-term average values. Along other boundaries, sufficient data coverage is available throughout the entire time period, though the list of stations providing data varies over time.

A consequence of assigning saltwater hydrostatic pressures to boundary cell faces along the southern two-thirds of the eastern boundary and along the southern boundary was that saltwater was presumed to occur at all elevations along these boundaries. The assignment of saltwater boundary heads had an appreciable effect on computed ground-water heads near the coasts of Biscayne and Florida Bays; however, no attempt was made to explicitly simulate a saltwater interface. Grid cells within model boundaries were considered to contain freshwater by virtue of the assignment of freshwater density values that remained invariant during simulations. An experiment was conducted in which some lower grid cells near the coast of Florida Bay were assigned saltwater densities. The effect on computed heads appeared to be negligible on a regional scale.

The southeastern corner of the grid (fig. 29) extended into part of the estuarine system. Because the aquifer continued into this area, the lower two model layers were left unmodified, and lateral saltwater boundary pressure were specified. The top (overland flow) layer cells in this region were eliminated from the solution. Cells in the second layer were adjusted in thickness to 0.1 ft, and vertical (overhead) boundary pressures were specified that were similar to monthly average stages measured at the Card Sound at Model Land Canal tidal station. These overhead boundary assignments largely controlled pressures computed in the underlying grid cells because they were located much closer to the position of the computed heads than the lateral boundaries.

Where controlled canals intersect the eastern coastal boundary (C-100, C-1, C-102, and C-103) after 1962, lateral no-flow boundaries are specified at the boundary cell face. This specifies that no flow exits the grid across the boundary face at the end of the canal string. Canal discharges at these locations were simulated as wells with rates equal to measured discharges. In time periods when discharge data were not available, time-period averages were used. This led to erratic stage fluctuations in canal grid cells adjacent to the boundary at certain simulation times when average discharge data were not compatible with prevailing wetter- or dryer-than-normal conditions. However, the effect was localized; in most of the modeled region, heads were not sensitive to the canal discharge specification.

Construction of Time-Period Models

Because the system of levees and controlled canals was developed in incremental stages through the 1945-89 simulation period, separate models are constructed for the five water-management time periods shown in figures 16, 17, 19, 20, and 21. Throughout each of these periods, the extent and operational use of manmade works remained generally unchanged. The extensive system of canals and controls shown in figures 20 and 21 was largely constructed over a period of several years before 1968; however, most of the canals were blocked until the latter part of 1967.

The time-period models were calibrated successively except for the first two, which were calibrated together because of the paucity of comparison data in the earliest time period. Generally, calibration of the first two time-period models, containing fewer water-management features, is based on adjustment of parameters describing the natural properties of the aquifer and surface-water flow systems and the natural processes of rainfall and evapotranspiration. Sensitivity analyses were performed to illustrate the degree of dependence of the simulations on these parameters. The parameter estimates receive additional testing in calibration of later time-period models, in which parameters describing the influence of the new canal and stage-control systems are determined. Sensitivity analyses are then performed in the later time-period models to assess the influence of the water-management system additions.

Hydraulic Conductivity Values and Layer Thicknesses

The hydraulic conductivity value assignments in each layer are shown in figure 30. Layer 1, the overland sheetflow layer, was assigned a uniform value of 3,000,000 ft/d to represent surface flow through vegetation and over a very rough surface, except for a section in the south where a higher value of 100,000,000 ft/d represented surface flow with less resistance through the lakes and channels of Taylor Slough. The value of 3,000,000 ft/d was determined by calibration. Data were insufficient for calibration of the higher value used in Taylor Slough, though with the higher value, simulated heads in surrounding areas agreed slightly better with data from a few scattered locations than when the lower value was used.

In parts of layer 2 corresponding to areas where Everglades peat or calcitic mud was present, low hydraulic conductivities of 10 ft/d represented the semiconfining effect of the materials (fig. 27). This value was large enough so that no appreciable difference between overland flow stage and the water-table altitude was computed. Otherwise, no known basis existed for calibration of this parameter value. Where the peats and marls are absent, layer 2 was treated as a vertical upward extension of layer 3, which is identified as the Miami Limestone everywhere except in the southeastern part of the grid where the Key Largo Limestone is present. Layer 3 was assigned a variable thickness that corresponds to that determined by Fish and Stewart (1991) on the basis of test drilling. The fourth and bottom layer corresponded to the Fort Thompson Formation and contemporaneous formations in the northeast and southeast and was similarly assigned a variable thickness based on results of the drilling program.

The distributed thicknesses of layers 3 and 4 are indicated in figure 31, a pair of surfaces that show the bottom elevations assigned to the third and fourth layers. As in figure 28, each intersection of lines represents the lateral center of a grid cell, and thin canal or

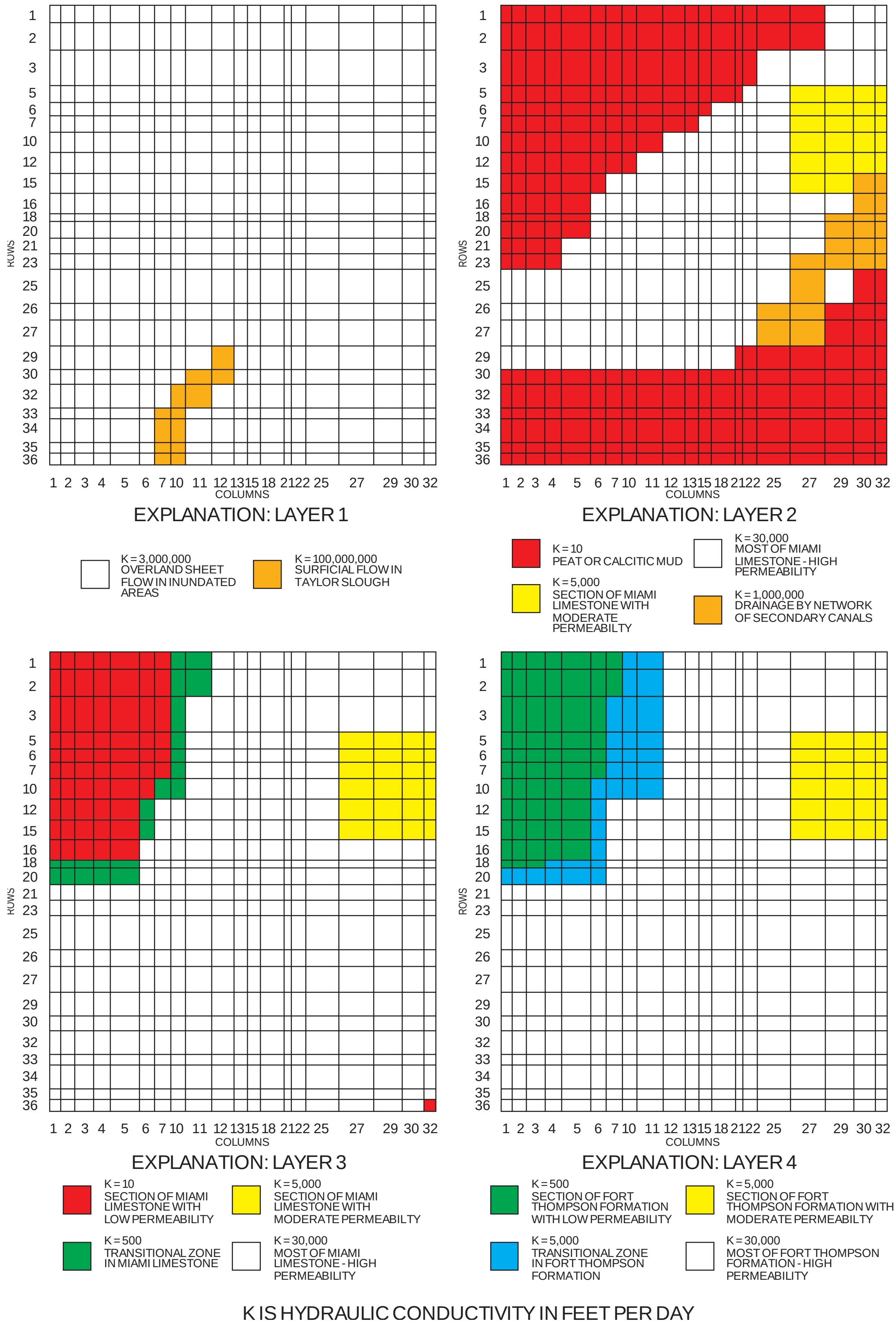


Figure 30. Distribution of assigned hydraulic conductivity values in each layer of the model.

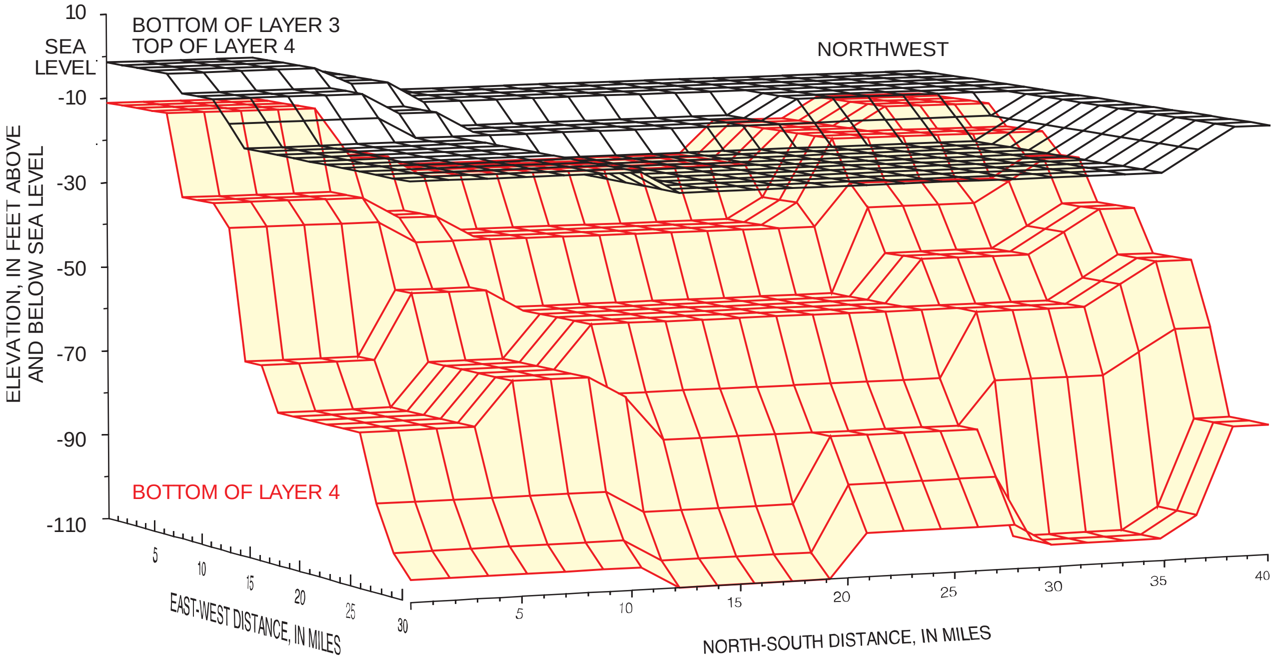


Figure 31. Three-dimensional representation of the bottoms of layers 3 and 4.

levee strings are omitted. The upper surface shows that layer 3 is thickest, extending to 25 ft below sea level, in the east-central part of the modeled area. To the southwest, the layer wedges out near land surface. Layer 4 deepens toward the eastern boundary of the modeled area, reaching a maximum depth of 110 ft. Toward the northwest and southwest, the bottom of the layer rises to within 10 ft below sea level. In the northwest, the thinning of layer 4 represents the wedging out of the Fort Thompson Formation.

Figure 30 shows where the horizontal hydraulic conductivity value of 30,000 ft/d was assigned after calibration to parts of layers 2, 3, and 4 representing the Biscayne aquifer. Vertical hydraulic conductivity was arbitrarily selected to be 3,000 ft/d. Because layers were less than 100 ft thick, a much lower value would not have affected the very small vertical flows. The high value of hydraulic conductivity (1,000,000 ft/d) assigned to an eastern part of the Biscayne aquifer segment of layer 2 represented the drainage effect of a vast network of secondary canals in an agricultural area; some calibration was required to arrive at the selected value. These canals are oriented in both the north-south and east-west direction.

Lower hydraulic conductivities were assigned to the northwestern parts of layers 3 and 4 to represent the reduced permeability of the thinning formations in this area, not considered to be part of the Biscayne aquifer. The sectors of low hydraulic conductivity are surrounded by thin transitional zones of moderate hydraulic conductivity (500-5,000 ft/d). Data are not available to support the existence of a transitional zone because the widely spaced wells of the surficial aquifer system study (Fish and Stewart, 1991) indicated either high or low formation permeability, but a gradational change in permeability was considered to be a more realistic description.

The northeastern section of lower permeability in the Biscayne aquifer, indicated by single-well aquifer tests (fig. 6), was represented by assigning a hydraulic conductivity value of 5,000 ft/d to cells in layers 2, 3, and 4 (fig. 30). This value and the boundaries of the section were adjusted in the calibration process.

General Calibration Procedure

Generally, calibration of the regional flow model in early time periods was accomplished by selecting values of porosity considered realistic, by using monthly rainfall totals from the Homestead Agricultural Experiment Station (fig. 26 and table 7) to describe recharge to the entire modeled area, and by varying aquifer hydraulic conductivities, maximum evapotranspiration rates, and the equivalent hydraulic conductivity of overland flow as calibration parameters. Shallow- and deep-root zone depths for the evapotranspiration calculation were also varied for calibration. The Homestead rainfall data were used because the station was located approximately in the center of the modeled region. Homestead Field Station data were used after 1986. Some adjustments were made to individual monthly rainfall rates when the measured Homestead values appeared unrepresentative for the area. The adjustments took the form of replacement of the Homestead monthly value with the monthly value from some other station, usually the Royal Palm Ranger Station (table 7), which was also approximately centrally located (fig. 26). The replacement value was also used for the entire region, except in time period 5. A list of rainfall value substitutions is included in table 10. In later time periods, adjustments were made to the equivalent hydraulic conductivity of canal nodes, and the representation of control structure operation (tables 5-6) was refined.

Because heads varied with the rise and fall of a free surface, the specified values for water and aquifer matrix compressibility coefficients (3 x 10^{-6} and 15 x 10^{-6} $(lb/in^2)^{-1}$, respectively) did not affect results of the calibration. (The computed change in storage, in this case, is dominated by the specific yield term, several orders of magnitude larger than the elastic compression term.) Sea-level atmospheric pressure (14.7 lb/in^2) was the specified base pressure used to determine the saturation depth in free-surface grid cells. The density of pure freshwater at 21° C was determined to be 62.3046 lb/ft^3 from standard references. Fluid within the model boundaries was assigned a slightly higher value (62.3216 lb/ft^3) because the native aquifer water has an average dissolved-solids concentration of about 300 mg/L. This value did not vary during the simulations. A boundary specification of seawater density (64.0 lb/ft^3) near the bay coasts was part of the mechanism of assigning saltwater hydrostatic boundary conditions, as previously described.

The assignment of parameter values, either as fixed estimates or as variables used to calibrate the model,was conservative in the sense that little or no attempt was made to specify detailed spatial variations. Spatially uniform values were specified for porosity (20 percent), rainfall, and maximum evapotranspiration rates. values, one corresponding to the coastal ridge (3 and 20 ft) and the other (0.1 and 5 ft) to wetlands where a surface layer of peat or marl was present. The deep-root zone depth is equivalent to the extinction depth. Hydraulic conductivity variations (fig. 30) were specified only in cases where substantial evidence indicated that major variations occurred between large sections of the modeled area.

The calibration of the model of the first time period began with a computation using time-period average monthly rainfall rates and boundary-head values. The purpose of this procedure was to make a transition from a specified spatially uniform initial head in the modeled region to a more realistic spatial distribution of heads that had partly converged to a pattern of seasonal variation. The mid-October head values after 1 year of the seasonal average computation were then used as initial conditions for the real-time computations used for calibration. Later time period models received inputs of the final pressures (or heads) computed in the previous time period, which were then used as an initial condition.

High rainfall recharge rates and high water-table altitudes during the summer months appeared to be coincidental with occasional instabilities in the computations that distorted the trend of computed heads. Smaller timesteps in these simulation periods reduced internodal fluxes and prevented these instabilities. For computational purposes, a year was considered to have 365.25 days and to be divided into 12 months of 30.4375 days each. Winter-month computational timesteps were 15.21875 days, and summer timesteps were 3.04375 days.

The maximum evapotranspiration rates (in inches per day) determined by model calibration are shown at the bottom of this page. These values are similar to the U.S. Weather Bureau pan evaporation rates in southern Florida for the 5-year period cited by Stewart and Mills (1967).

The specified effective porosity of 20 percent was slightly less than that indicated by the comparison of rainfall and water-level changes in a well near Homestead (table 1). An effective porosity of 1.0 was assigned to flowing surface water. These values remained unchanged in the calibration of the time-period models and were only varied in sensitivity analyses.

January	February	March	April	May	June-October	November	December
0.08	0.11	0.14	0.17	0.18	0.21	0.12	0.11

Table 10. Monthly rainfall values substituted for principal measuring station values for model calibration

[Rainfall values in inches. **Bold** indicates substituted values; dashes (—) indicate data not measured]

Date	Homestead[1] Agricultural Experiment Station	Royal Palm Ranger Station	Tamiami Trail at Forty-Mile Bend	Miami Interna-tional Airport	Dade-Broward Levee	Trail Glades Range	Homestead[1] Field Station	NP-206	NP-203	Chekika State Recreation Area	S-331
Time Period 1 (Water Years 1945-52)											
09/52	5.37	5.48	**7.45**	4.44	13.21	—	—	—	—	—	—
Time Period 2 (Water Years 1953-61)											
05/58	18.60	**9.45**	11.72	18.14	—	—	—	—	—	—	—
03/59	7.27	**4.87**	5.57	4.85	4.28	—	—	—	—	—	—
07/59	10.59	**7.86**	6.29	8.20	7.47	—	—	—	—	—	—
09/59	10.32	**7.21**	8.61	12.98	—	—	—	—	—	—	—
11/59	9.72	**5.43**	5.58	13.15	4.70	—	—	—	—	—	—
06/60	13.52	**11.17**	8.86	6.31	7.96	—	—	—	—	—	—
07/60	13.90	**10.56**	—	5.27	8.05	—	—	—	—	—	—
09/60	19.04	**15.33**	19.05	24.40	—	—	—	—	—	—	—
10/60	6.79	**5.49**	7.23	10.54	—	—	—	—	—	—	—
Time Period 3 (Water Years 1962-67)											
05/64	2.22	**5.80**	4.37	4.67	4.50	—	—	—	—	—	—
06/65	5.86	**7.21**	7.24	6.55	—	—	—	—	—	—	—
07/65	4.14	3.03	7.55	**6.56**	—	—	—	—	—	—	—
Time Period 4 (Water Years 1968-82)											
06/68	23.98	**16.78**	14.11	22.36	—	—	16.57	—	—	—	—
06/71	5.86	**10.70**	6.81	11.65	—	—	11.34	—	—	—	—
09/76	12.46	**8.82**	6.76	7.75	—	5.40	—	—	—	—	—
06/80	17.19	14.61	4.13	3.02	—	—	**10.50**	—	—	—	—
07/80	12.43	12.37	4.36	9.40	—	—	**9.02**	—	—	—	—
06/81	4.62	3.47	4.13	5.49	—	—	**8.02**	—	—	—	—
09/81	21.57	13.01	8.65	14.79	—	—	**13.13**	—	—	—	—
07/82	4.37	**5.50**	7.04	3.84	—	2.20	—	—	—	—	—
08/82	5.03	**8.56**	4.35	5.79	—	6.30	—	—	—	—	—
09/82	4.37	7.85	4.16	7.62	—	9.30	**9.52**	—	—	—	—
Time Period 5 (Water Years 1983-90), Region 1 - Eastern Half											
05/85	3.21	**5.75**	—	—	—	—	4.19	—	7.13	5.58	5.52
06/86	2.93	**3.63**	—	—	—	—	3.20	2.75	7.05	7.23	9.50
09/86	9.19	**4.53**	—	—	—	4.40	8.41	—	5.15	3.31	3.17
09/88	4.70	5.20	6.11	—	—	2.60	5.22	2.90	4.81	**3.76**	1.09
Time Period 5 (Water Years 1983-90), Region 2 - Western Half											
05/85	3.21	5.75	—	—	—	—	4.19	—	**7.13**	5.58	5.52
06/85	2.93	3.63	—	—	—	—	3.20	2.75	**7.05**	7.23	9.50
07/85	11.44	11.82	11.74	—	—	—	11.69	15.22	**12.05**	8.26	10.09
08/85	4.80	5.32	6.74	—	—	—	6.68	6.04	**9.31**	7.06	5.59
09/85	6.57	7.28	—	—	—	—	8.57	2.75	**6.44**	8.22	8.55
06/86	11.23	—	—	—	—	6.60	12.23	13.23	**12.34**	14.01	8.82
07/86	2.39	5.15	4.38	—	—	—	3.96	8.34	5.58	**9.85**	2.27
08/86	6.12	7.90	—	—	—	5.90	3.89	3.78	6.42	**8.41**	8.92
09/86	9.19	4.53	—	—	—	4.40	8.41	—	**5.15**	3.31	3.17
05/87	6.87	8.10	3.69	—	—	1.90	4.23	6.42	**4.52**	4.38	2.56
06/87	2.63	1.30	2.43	—	—	7.40	4.68	—	—	**6.90**	13.01
07/87	5.44	4.15	8.42	—	—	5.80	6.99	3.75	—	**6.75**	4.23
08/87	3.55	6.82	4.61	—	—	9.30	5.85	8.89	—	**9.07**	6.44
09/87	10.22	10.23	12.03	—	—	9.10	9.44	13.31	**11.75**	10.81	12.33
09/88	4.70	5.20	6.11	—	—	2.60	5.22	2.90	4.81	**3.76**	1.09

[1]The Homestead Agricultural Experiment Station was the principal measuring station until 1987 when the Homestead Field Station became the principal measuring station. Details of the rainfall stations are given in table 7 and locations are shown in figure 26.

As previously indicated, the extinction depth, below which the evapotranspiration rate for the ground-water reservoir is zero, was arbitrarily set equal to 20 ft in the noninundated part of the modeled area, despite the fact that the soil zone in much of southern Dade County is only a few inches thick and is underlain by hard limestone rock. However, the water-table altitude drops steeply during periods without rainfall throughout the area even when the water table is more than 10 ft below land surface (for example, S-196A in March 1956, pl. 2). The lack of apparent correlation between rate of recession and depth below land surface suggests a large extinction depth. On the other hand, it was found during calibration that an extinction depth of 20 ft caused computed heads in the peat and marl soil regions to drop far below observed heads during dry periods. In actuality, the peat and marl soils retain considerable moisture just below the surface even during extended dry periods, and the appropriateness of the evapotranspiration algorithm is questionable in the glades regions. To obtain more realistic simulated water-table behavior in these regions during dry periods, the extinction depth (bottom of the deep-root zone) for the glades region was set at 5 ft in the calibrated model.

Early efforts by the author to calibrate the model using a different set of hydraulic conductivities and porosities than those just cited were apparently successful. The surface layer of the aquifer (layer 2 in the model) was assigned an effective porosity of 20 percent, as in the final calibrated version, but underlying permanently saturated layers were assigned a higher value of 35 percent based on neutron porosity logs run in wells penetrating the Upper Floridan aquifer. A hydraulic conductivity of 5,000 ft/d was assigned to the Miami Limestone (layer 3 and much of layer 2) on the basis of some single-well aquifer tests (Fish and Stewart, 1991), the observed infilling of some solution pores by sand, and the frequent success of aquifer dewatering operations in some parts of the study area. A hydraulic conductivity of 40,000 ft/d was assigned to the highly permeable part of the Fort Thompson Formation (layer 4). Vertical hydraulic conductivities were less by an order of magnitude in both layers.

A concurrent effort to simulate the chloride plume originating from the flowing artesian well in Chekika State Recreation Area (Merritt, 1993) yielded results inconsistent with some measured data when these hydraulic coefficients were used. The extent of the simulated plume in the Fort Thompson Formation (layer 4) was in agreement with surface resistivity and water-quality measurements (Waller, 1982). However, the extent of the simulated plume in upper layers was smaller, indicating substantial vertical stratification that was inconsistent with the measured water-quality data.

The assigned value of porosity was made vertically uniform at 20 percent, reflecting a new assumption that specific yield estimates for surface rocks near Homestead could serve as a reasonable estimate of the effective porosity of the Biscayne aquifer throughout its full thickness. The hydraulic conductivity of all highly permeable parts of the Biscayne aquifer, excluding the northeastern block, was made vertically uniform at a value of 30,000 ft/d. Previously cited evidence of lower permeability in the Miami Limestone was considered to be characteristic either of only the extreme northeastern part of the modeled area or of a thin upper layer of rocks in a larger area. Without revising any other parameter assignments, an equally good flow model calibration was achieved, and the corresponding plume model produced an excellent simulation in all layers. Using these parameter values, heads throughout the modeled region were approximately simulated in the 1945-89 water years (pls. 1-9). Further details of the calibrations are presented in the sections describing the use of the models in each of the five time periods for various interpretations related to the development of the water-management system.

Evaluation of Simulation Results

A discussion of sensitivity analyses and water-budget analyses is presented in the following pages to illustrate the behavior of the model and the natural flow system that it represents and to identify the parameters that function as primary natural controls on the flow system and water-table altitudes. This is followed by detailed interpretations of the calibrated time-period models and their application.

Natural Process Sensitivity Analyses

The author's approach to sensitivity testing is to avoid comparing the sensitivity of the model to one parameter with that of the model to another parameter. Rather, in the sensitivity tests, perturbations of parameter estimates based on measured data are similar in scale to a realistic range of possible measurement error, so that the sensitivity of the model to parameter estimates affected by measurement error can be tested. Perturbations of parameters that are determined by model calibration, on the other hand, are useful in assessing the precision with which the model has determined that parameter. Plate 1 shows results of sensitivity analyses performed to demonstrate how the model operates and to assess the influence on aquifer flows of the various parameters and corresponding hydrologic processes. Computed hydrographs based on parameter values revised for the sensitivity analyses are shown as dashed, dotted, or chain-dotted lines in the time period 2 (water years 1953-61) hydrographs.

Slight differences might exist between hydrographs used as control for the sensitivity analysis and the calibration curves because minor modifications to modeling procedures were made after the sensitivity analyses were completed. Any such differences have no significance for the results of the study.

Aquifer Hydraulic Conductivity and Porosity

Heads computed in sensitivity analyses were plotted against the calibrated model heads at all observation wells and gaging stations shown on plate 2. Well S-196A, north of Homestead, was selected as likely to show a typical response of computed heads to perturbations in the calibrated value of hydraulic conductivity of the principal part of the Biscayne aquifer because of its central location on the southern coastal ridge. The calibrated value of 30,000 ft/d was increased and decreased by 50 percent. The hydrograph (pl. 1) shows that during a period of high water table, a value of 15,000 ft/d increased the computed water level at the well by as much as 1 ft, and a value of 45,000 ft/d decreased the computed water level by as much as 0.4 ft. During periods when the water table was low, computed water levels based on the three estimates were more nearly the same. The value of 15,000 ft/d appeared to raise the S-196A water level too high, especially during wet periods. The similarity of the hydrograph obtained using a value of 45,000 ft/d to the calibration hydrograph suggested that model calibration probably cannot clearly discriminate among hydraulic conductivity values falling within a range of 25,000 to 50,000 ft/d or higher. The simulated water table, therefore, appears to be somewhat insensitive to large changes in the value of hydraulic conductivity in the range of its probable value. Because of the high transmissivity of the aquifer and low head gradients, the hydraulic conductivity variations do not cause significant head variations.

Well S-196A is also considered typical in the response of its computed water level to variations in the porosity estimate based on rainfall and water-level measurements leading to estimates of specific yield (table 1). Effective porosity was decreased by 50 percent to 10 percent and increased by 75 percent to 35 percent (pl. 1). This is greater than the likely possible error in the specific yield estimate, which was cited as falling in the range of 20 to 25 percent. The corresponding effects of the porosity changes were to magnify and dampen the response of the computed water level to seasonal changes in the rate of recharge and evapotranspiration. However, the variations in computed water level were moderate. Wet-season peaks appear too high when porosity is 10 percent, exceeding the calibrated model peaks by as much as 1.3 ft. Dry-season minima appear too low in most years. When porosity was 35 percent, differences with the calibrated model are more moderate. Wet-season peaks are as much as 0.5 ft lower than the calibrated curves, and dry-season minima are about 0.5 ft higher. The calibrated model porosity value (20 percent) is clearly a better choice than the two tested alternatives.

Equivalent Hydraulic Conductivity of Wetlands

The response of the model to changes in the value of equivalent hydraulic conductivity assigned to the overland flow layer in the Everglades and southern glades was assessed by comparing hydrographs of computed stages at gaging stations and wells located in wetlands areas. The equivalent hydraulic conductivity value also determines the rate of surface flow across model boundaries where the specified head values are above land surface. Everglades surface-water station P-33 in the upper part of Shark River Slough provides a typical example of the changes in computed stage (pl. 1). The calibrated model value of 3,000,000 ft/d is a small fraction of the value estimated earlier in this report using equation 6. This value would result from using a Manning’s roughness coefficient of 30.0 in equation 6, about 60 times greater than most values of Manning’s coefficient reported in the literature. This can possibly be explained by observing that the overland flow encounters substantial resistance from vegetation and from the irregularity of land surface. In this respect, overland flow shares some of the characteristics of groundwater flow (subdivision into multiple pathways, lack of turbulence) (Maheshwari, 1992) through solution features in rock. These observations lend support to the use of the Darcian algorithm for computing the rate of overland flow in the wetlands region.

The stage at P-33 was higher than land-surface elevation during most of the period from 1953 to 1955 and from mid-1957 to 1961, so that computed surface-water stage provides a good measure of model sensitivity to equivalent hydraulic conductivity. The stage rarely exceeded 2 ft above land surface. In the sensitivity analysis, the calibrated value of equivalent hydraulic conductivity was increased and decreased by 50 percent (to 4,500,000 and 1,500,000 ft/d) and increased by an order of magnitude to 30,00,000 ft/d. When 1,500,000 ft/d was the assigned value, the simulated September 1960 peak stage was increased by about 0.4 ft, and when 4,500,000 ft/d was the assigned value, the simulated peak stage was decreased by about 0.2 ft. When 30,000,000 ft/d was the assigned value, the simulated September 1960 peak stage decreased about 0.45 ft. Thus, the surface-water stage seems to be relatively insensitive to moderate changes in the equivalent hydraulic conductivity, and the model calibration using well water levels and gaging-station stages does not determine this value with precision. Equivalent hydraulic conductivity values ranging from 1,500,000 to 4,500,000 ft/d yield simulated stages at P-33 that could be accepted as a calibration of the model.

Computed flows at the location of P-33 were compared in the three cases (equivalent hydraulic conductivities of 1,500,000, 3,000,000, and 4,500,000 ft/d) at several times during 1953 and 1954. The rates of flow for all nondry overland-flow grid cells were averaged. At a simulation time corresponding to July 2, 1953, when the simulated stage at P-33 agreed with the measured stage of about 0.8 ft above land surface, the average overland-flow velocities for the three cited runs were 89, 173, and 221 ft/d, respectively. Values in some individual grid cells had an even wider range of variation. This demonstrates that the computed overland flow rates are highly sensitive to moderate variations in the value specified for equivalent hydraulic conductivity. An important conclusion of this analysis is that the calibration of the model using water levels in wells and surface-water stages has not determined overland flow rates with a degree of accuracy that would be considered sufficient for many purposes. Additional calibration procedures, such as the use of measured overland flow rates, would be required to test the capability of the model to accurately simulate rates of overland flow.

When a computational timestep of 15.21875 days was used, oscillatory instabilities began to occur when equivalent hydraulic conductivity values increased to more than 75,000,000 ft/d in the entire overland flow region. The use of coefficient values of 100,000,000 ft/d in the Taylor Slough area, however, did not seem to cause any instability problems. These high values, chosen arbitrarily, enabled Taylor Slough to act as a simulated drain for the overland flow layer, and may have affected the choice of the value of 3,000,000 ft/d used in the remainder of the overland flow layer.

Rainfall and Evapotranspiration

Sensitivity analyses were also designed to assess the sensitivity of the calibrated model heads to variations in rainfall and evapotranspiration rates. In the rainfall analysis, rainfall specifications based on measured monthly values in water years 1953-61 were increased and decreased by 20 percent. Plate 1 shows comparison hydrographs from S-196A in the southern coastal ridge, G-612 in the eastern coastal lowlands, and G-620 in the seasonal sheetflow region on the northeastern boundary of Shark River Slough. The 20 percent changes produce substantial computed water-level changes at S-196A, averaging 1.2 to 1.5 ft throughout most of the time period. However, at G-612, where natural water-level variations were only about one-third those measured at S-196A, the 20 percent variations in rainfall cause water-level changes of only about 0.5 to 0.75 ft. Overland flow stages at G-620 increased about 0.4 to 0.8 ft when the higher rainfall values were used. When rainfall amounts were decreased 20 percent, the site was dry about 50 percent of the time.

Another type of rainfall variation was tested by representing a hypothetical rainfall “spike.” This sensitivity analysis was more directly related to the type of data error that might actually occur, as if a highly localized storm caused a measured value to be unrepresentative of a larger area. The monthly rainfall for September 1955 was increased by 10 in. above the 8.75 in. specified in the calibrated model. Effects at the three observation sites are shown on plate 1. The S-196A hydrograph shows that the water level at the end of September 1955 is 2.5 ft higher than the value in the calibrated model. The subsequent water-level decline is steeper, and by the end of the subsequent recession period, in July 1956, the calibrated and “spiked” water levels differ by only about 0.15 ft. Similar effects of lesser magnitude are shown at G-612 and G-620. This test shows the potential effect on the simulation of substantial errors in the measurement and specification of rainfall. In summer months with intense rainfall, recorded monthly totals can easily differ by 5 to 10 in. at stations separated by several miles. Therefore, errors in the measurement of rainfall can substantially affect the heads computed by the model, and are a primary source of error for a regional model of surficial aquifer flow of the kind developed as part of this study. This sensitivity analysis can also be interpreted to assess the effect of a "wet" tropical storm or hurricane. Such storms have dropped as much as 20 in. of rainfall in the study area.

Results of tests in which the maximum evapotranspiration rate was increased and decreased by 20 percent (pl. 1) were similar to those when rainfall amounts were changed by 20 percent. Because the specified maximum evapotranspiration rates had less variability than the monthly rainfall data, the computed water-level changes relative to the calibrated model are more uniform in time. The general variation at S-196A is about 1.2 to 1.5 ft as in the rainfall tests. The variation at G-612 (not shown) is similarly dampened, and variations at G-620 are similar to those of the rainfall analysis. This analysis demonstrates that the maximum evapotranspiration rate was determined relatively precisely by the calibration. Because the calibrated rates were similar to measured pan evaporation rates, the latter probably could have been used without being a source of error.

As previously indicated, calibration of heads in inundated and noninundated parts of the modeled area required regionalization of the extinction depth (bottom of the deep-root zone). To illustrate the effect on simulated heads, two sensitivity analyses were made. In the first, the glades extinction depth of 5 ft was assigned uniformly to the entire modeled region, including the coastal ridge. The result is shown at well S-196A in the coastal ridge on plate 1. The assumption of a 5-ft extinction depth at this location appears untenable unless other parameter values are substantially revised. Computed heads range from 2.5 to 4.0 ft above those of the calibrated model. In addition, the annual range of variation is too shallow to match observed data.

A second sensitivity analysis assumed that the coastal ridge extinction depth (20 ft) also applied to the glades regions. The result is shown on plate 1 for the grid cell corresponding to G-620. When water is above land surface, the computed stage matches that of the calibrated model. However, when the area is shown by the model to become dry, the computed head drops precipitously far below measured heads. When the extinction depth is 5 ft, the head decline generally is realistic, though a close match of the measured data is usually not achieved. The data suggest a qualitative difference in the mechanism of subsurface drying between the coastal ridge limestone rocks and the glades peats and marls.

It might seem that increasing the rate of drainage (by varying hydraulic conductivity and porosity specifications) would be equivalent to varying maximum evapotranspiration rates or extinction depths in matching observed recession curves. The rate of drainage, however, has diverse seasonal patterns of variation inland and near the coast and near and far from drainage canals. Additionally, the range of head variation in recession periods achievable by realistic variations of hydraulic parameters is limited. Realistic variations of evapotranspiration and hydraulic parameters tend to have qualitatively different effects on computed seasonal head variations in different parts of the modeled area that become evident when attempting to match observed data.

Mass-Flux Analyses

Another method of examining the relative significance of flow processes in southern Dade County is to consider the relative masses of water originating from various sources at selected locations in the area. The author's version of the SWIP code facilitates such an analysis by providing an annotated summary of water, solute, and thermal fluxes in a designated vertical column of the grid. Flux summaries were obtained for vertical columns of grid cells corresponding to the approximate areal locations of rocky glades well G-1502, Atlantic Coastal Ridge wells S-196A and G-613, Everglades coastal ridge (Long Pine Key) well NP-44, eastern coastal lowland well G-612, Shark River Slough surface-water station P-33, and southern glades surface-water station P-37. (Table 2 and figure 17 can be used to determine map locations. Well S-524 indicates the location of G-1502 in fig. 17.)

Table 11 lists the average horizontal water mass fluxes, the net horizontal fluxes, and the vertical water mass fluxes in a column of unit length and width at the selected locations. Values, in pounds per day, are tabulated for each layer in the model, except that fluxes for vertically adjacent layers of the same permeability are lumped together as indicated. Values associated with "*A*" and "*N*" symbols are the average and net variation

Table 11. Water mass flux through a column of unit length and width at selected locations

[Values are in pounds per day and can be converted to cubic feet per day by dividing by the density of freshwater (about 62.4 pounds per cubic foot). Net atmospheric flux is rainfall minus evapotranspiration per unit area. Arrows show vertical flux per unit area; dashes (--) indicate data not included when surface layer is dry. Abbreviations: N, net horizontal flow; A, average horizontal flux]

Station	Budget/layers	Layer thickness (feet)	Hydraulic conductivity (feet per day)	Low water (05/15/53)	Rapid increase (06/16/53)	High water (10/30/53)	Mid-recession (12/15/53)
G-1502	Net atmospheric flux			-0.45	+1.93	+0.30	-0.44
	Overland flow layer	--	--	Dry	Dry	Dry	Dry
	Layers 2-4	55.0	30,000	N = +0.20, A = 6,219	N = +0.03, A = 7,044	N = -0.19, A = 1,312	N = +0.19, A = 2,219
S-196A	Net atmospheric flux			-0.55	+1.81	+0.30	-0.46
	Overland flow layer	--	--	Dry	Dry	Dry	Dry
	Layers 2-4	76.0	30,000	N = +0.19, A = 5,754	N = +0.21, A = 4,706	N = -0.22, A = 11,273	N = -0.44, A = 10,542
G-613	Net atmospheric flux			-0.61	+1.75	+0.30	-0.46
	Overland flow layer	--	--	Dry	Dry	N = +0.11, A = 404	Dry
	Layers 2-4	59.0	30,000	N = +0.21, A = 1,649	N = +0.21, A = 1,824	N = -0.25, A = 7,299	N = +0.07, A = 5,290
NP-44	Net atmospheric flux			-0.56	+1.81	+0.30	-0.46
	Overland flow layer	--	--	Dry	Dry	Dry	Dry
	Layers 2-4	45.0	30,000	N = +0.08, A = 2,003	N = +0.10, A = 1,811	N = -0.25, A = 3,262	N = +0.07, A = 2,567
G-612[1]	Net atmospheric flux			-0.62	+1.70	+0.30	-0.46
	Overland flow layer	--	--	Dry	Dry	Dry	Dry
	Layer 2 (canal)	7.5	300,000,000	N = +3.20, A = 27,930	N = +16.4, A = 466,055	N = -793.3, A = 2,975,788	N = -506.7, A = 1,535,190
	Layers 3-4	75.0	30,000	N = +5.30, A = 112	N = -15.1, A = 1,412	N = +111.9, A = 4,584	N = +80.0, A = 4,164
P-33[2]	Net atmospheric flux			-0.47	+2.00	+0.30	-0.46
	Overland flow layer	9.0	3,000,000	Dry	Dry	N = -0.19, A = 8,335	N = +1.57, A = 6,861
	Layers 2-3	18.0	10	N = +0.01, A = 0.46	N = 0.00, A = 0.30	N = 0.00, A = 0.54	N = 0.00, A = 0.54
	Layer 4	10.0	500	N = -0.07, A = 31	N = -0.03, A = 20	N = -0.47, A = 53	N = -0.49, A = 55
P-37[3]	Net atmospheric flux			-0.62	+1.70	+0.30	-0.46
	Overland flow layer	13.4	3,000,000	N = +0.38, A = 958	N = +1.63, A = 3,422	N = -0.60, A = 39,153	N = -0.51, A = 16,895
	Layer 2 (calcitic mud)	.6	10	N = 0.00, A = 0	N = 0.00, A = 0	N = 0.00, A = 0	N = 0.00, A = 0
	Layers 3-4	30.0	30,000	N = -0.41, A = 895	N = -0.93, A = 2,033	N = +0.63, A = 1,369	N = +0.41, A = 883

[1]For G-612, the four values representing the vertical flux per unit area between layers 2 and 3 are: ↑5.3, ↓15.1, ↑111.9, and ↑80.1.

[2]For P-33, the four values representing the vertical flux per unit area between the overland flow layer and layer 2 are: 0.00, 0.00, ↓0.47, and ↓0.49. Between layers 3 and 4, the values are: ↓0.07, ↓0.02, ↓0.47, and ↓0.49.

[3]For P-37, the four values representing the vertical flux per unit area between the overland flow layer and layer 2 are: ↓0.41, ↓0.93, ↑0.63, and ↑0.41. Between layers 2 and 3, the values are ↓0.41, ↓0.93, ↑0.63, and ↑0.41.

of horizontal mass fluxes within the layer or group of layers. Vertical mass fluxes into layers of different permeability are indicated by arrows showing direction of the movement. Horizontal fluxes in the overland flow layer are shown at some computation times for P-33, P-37, and G-613. Canal (layer 2) fluxes are shown for G-612. The net atmospheric flux is rainfall recharge minus evapotranspiration, in pounds per day, divided by the surface area of the column of grid cells. At any given time, these values can vary spatially because computed evapotranspiration rates vary with depth to the water table. The values may be the same at different locations, however, when the water table is within the shallow-root zone at those locations.

The net horizontal flux (*N)* was computed as the algebraic sum of the signed horizontal flux components across the grid cell faces in the x- and y-coordinate directions, divided by the surface area of the column of grid cells. A positive sign in table 11 denotes a net gain and a negative sign denotes a net loss. *N* can be considered mathematically as the divergence of a flux vector ($\nabla \bullet F$) within the local plane of the model layer. The average flux (*A*) was calculated as the square root of the sum of squares of the average x- and y-direction fluxes per unit length and width. It was also calculated from the resultant horizontal velocity vector length at the grid cell center (computed routinely by the author's version of SWIP), by multiplying the velocity by water weight density (62.4 lb/ft^3) and layer thickness. Values calculated by the two methods were identical except when a layer was only partially saturated. The arithmetic procedures were automated within the SWIP code and checked with manual calculations. The flux analysis was performed for four separate computational times within the 1953 calendar year, representing the annual low water level, the early wet-season buildup, the annual high water level, and the dry-season recession.

As an example of this computation, the computed average mass flux (*A*) at S-196A on May 15, 1953, is 5,754 lb/d (pounds per day). This is the estimated amount of water that passes horizontally through a vertical column of unit length and width and of thickness corresponding to the sum of the saturated thicknesses of layers 2, 3, and 4, at this location. Layer 2 contains the free water surface. If the differences in flux across the column in the x and y direction are summed, the result is the net horizontal flux (*N*) within the column, which is 0.19 lb/d in this example. The overland flow layer is dry in this example. If the estimated evapotranspiration rate for the column of grid cells is subtracted from the specified rainfall recharge rate and divided by the surface area of the column of grid cells, the result (-0.55 lb/d) is the net atmospheric flux per unit area. The negative sign indicates that the evapotranspiration rate exceeded the recharge rate. At gaging station P-37 on the same date, downward flux within the vertical column, divided by the surface area of the column of grid cells, was 0.41 lb/d from the overland flow layer into the peat and marl layer (layer 2) and was 0.41 lb/d from the peat and marl layer into the Biscayne aquifer (layers 3 and 4).

At the rocky glades well (G-1502) and the coastal ridge wells (S-196A, NP-44, and G-613), net horizontal fluxes are small, less than 0.015 percent of the average horizontal flux. The horizontal flux computations include the model layer containing the free water surface. Net atmospheric fluxes are also very small, less than 0.10 percent of the average horizontal flux through the underlying water column. However, net atmospheric fluxes are usually substantially higher than the net horizontal fluxes, which helps to explain why heads computed by the model are especially sensitive to variations in rainfall and maximum evapotranspiration rates. The net atmospheric flux is shown to be greatest relative to net horizontal flux in the early wet season when water-table altitudes increase rapidly. In late October, at the beginning of the dry season, however, the net horizontal fluxes are negative and almost equal in magnitude to the net atmospheric flux. This indicates a higher rate of drainage that is a somewhat delayed response of the inland ground-water flow system to wet-season recharge.

The grid cell closest to well G-612 was located in a string of canal nodes representing the Florida City Canal. An examination of fluxes in this vertical column provided an analysis of the typical pattern of ground-water fluxes computed by the model in the vicinity of a canal. As at inland well sites, the average horizontal flux is large compared to the net horizontal flux and to the net atmospheric flux. However, the net horizontal flux in the aquifer grid cells is many times larger than at inland sites and is nearly identical to the vertical upward flux from the aquifer cells into the canal cell. This demonstrates that the canal gains substantial volumes of water from the aquifer. The large horizontal fluxes shown for the canal cell suggest that the canal discharges substantial flow at the eastern boundary compared to ground-water flow paths of a similar cross-sectional area. On the dates shown in table 11, the head difference between the aquifer and the canal cells did not exceed 0.01 ft.

A dissimilar pattern of mass fluxes is evident at the nodal location corresponding to P-33. Here, layers 2 and 3 are relatively impermeable and have very little flow, but modest flow occurs in a thin fourth layer, which is assigned a hydraulic conductivity value less than 2 percent that of the aquifer in the coastal ridge area. When overland flow occurs, a net horizontal flow loss in the fourth layer is balanced by downward percolation at the same rate from the surface overland flow layer. Fluxes in the overland flow layer in October and December seem relatively small, but the computed depth of flow varies from only about 0.75 ft in October to 0.45 ft in December. The overland flow velocity computed by the model in the vicinity of P-33 was about 125 ft/d at both computational times. The head difference between the overland flow layer and the fourth layer was about 0.01 ft.

The pattern of computed mass fluxes in the cell representing P-37 is similar to that for P-33, except that fluxes in layer 4, a 30-ft thick section of the Biscayne aquifer, are comparable to those at the coastal ridge well locations. Another dissimilarity is that fluxes through the layer of low permeability between the aquifer and the layer of overland flow are upward during the wet season and downward during the dry season and balance flux gains and losses from the Biscayne aquifer layers. This suggests a pattern of upward and downward seepage between the aquifer and the body of flowing surface water. The computed overland flow depth varies from about 0.2 ft in May to about 1.1 ft in October. Computed overland flow velocities in the vicinity of P-37 vary from about 30 ft/d in May to about 540 ft/d in October. The head difference between the overland flow layer and layer 3 did not exceed 0.001 ft on the four dates shown in table 11.

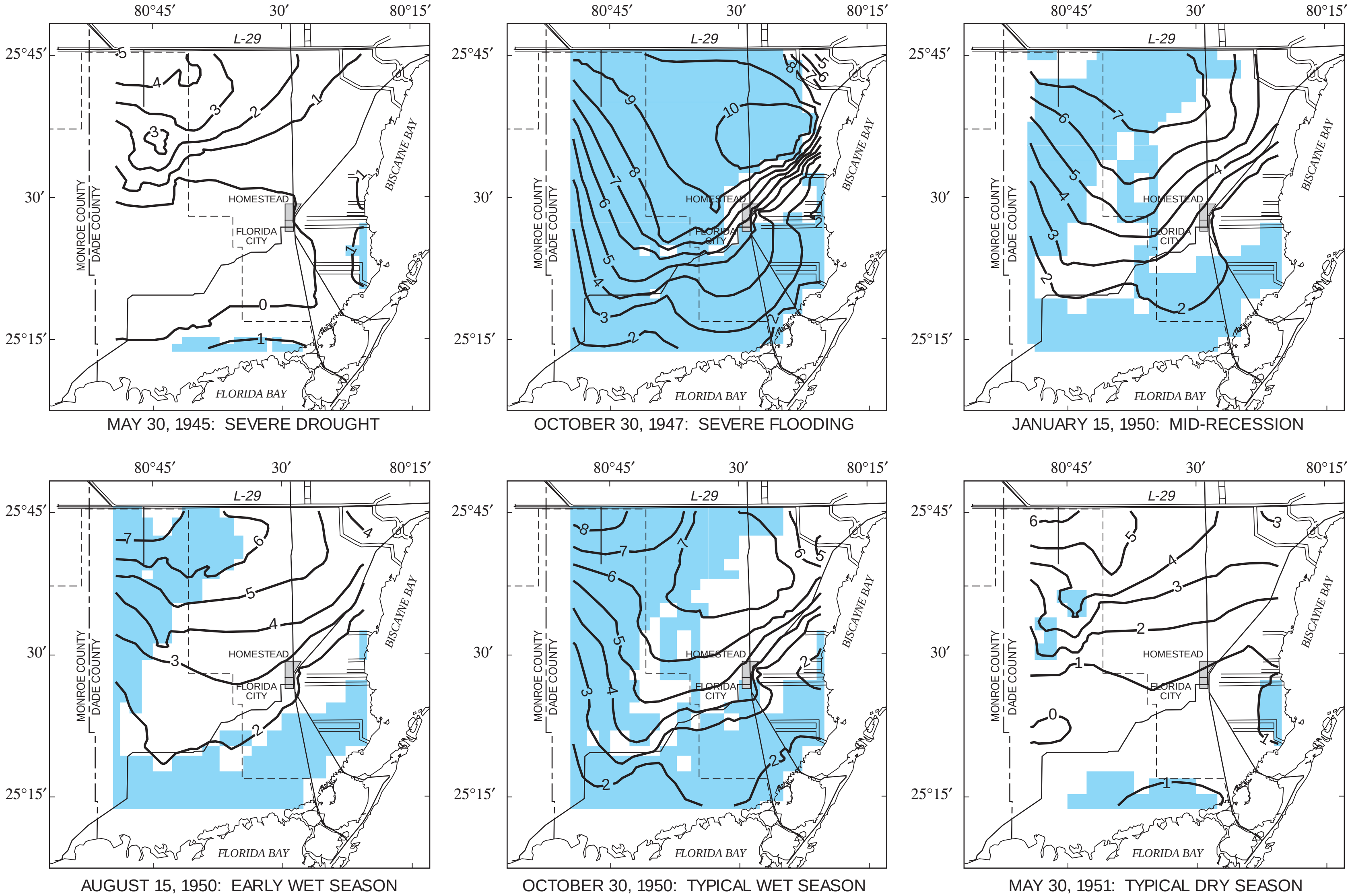
Another facet of the seasonal behavior of the flow system in the modeled area is observed by comparing average horizontal aquifer fluxes at G-1502, S-196A, G-613, and G-612 computed for the four times given in table 11. Average horizontal fluxes at G-613 and G-612, which are nearest to coastal discharge points, are highest in October, at the time of highest water. Average horizontal ground-water fluxes at S-196A, farther upgradient, are highest between October and December. At G-1502, near the area where the altitude of the wet-season water table is highest in 1953, the highest fluxes are in May and June during the previous dry season. This indicates a delayed drainage effect in which the most rapid drainage following wet-season recharge occurs nearest the discharge boundaries for the flow system. Maximum drainage at locations more distant from the boundaries may be delayed for several months, and at the farthest inland locations, may not occur until the following dry season.

Evaluation of the Simulation for Time Period 1 (Water Years 1945-52)

The model appears to overestimate measured heads in 1947 and 1948 (pl. 1). It has been observed that the input data errors causing the most serious discrepancies during model calibration were nonrepresentative wet-season monthly rainfall totals, and that some calibration problems were apparently resolved by substitution of rainfall totals from another rainfall station in the region. This procedure was not followed for the 1947-48 summer months because data were available from only two other stations, and those data did not justify substitutions. During the 1945-52 time period, the only substitution for a Homestead Agricultural Experiment Station rainfall value (table 10) was that for the final month, September 1952, for which a higher value from the rainfall station on the Tamiami Trail at Forty-Mile Bend was selected.

Dry- and Wet-Season Comparisons

Near the beginning of the first time period in early 1945, one of the most severe droughts of record occurred (Douglas, 1947). Using rainfall data from the S-196A well site, the lowest 1945 water level in S-196A depicted by the model is more than 1 ft higher than the recorded minimum, possibly because of the monthly averaging of model input parameters or the overspecification of antecedent rainfall. Nonetheless, a severely depleted aquifer is depicted by the computed water-table contours shown in figure 32 for May 30, 1945. (The contours are drawn from ARC/INFO TIN lattices generated from model-computed heads, and the heads are also compared with specified land-surface elevations to estimate the area inundated.) A large region in the southwestern part of the county, including much of future Everglades National Park, is shown to have had a water table lower than sea level. According to the simulation, the only areas inundated on May 30, 1945, occurred in narrow strips adjacent to model boundaries that represent the coasts of Biscayne and Florida Bay. Specified coastal boundary heads were higher than those below sea level computed for the inland region.



EXPLANATION

0 5 MILES
0 5 KILOMETERS

INUNDATED AREA—Shaded area is restricted to area contained within the model area

—2— SIMULATED WATER-TABLE CONTOUR—Shows altitude of water table. Contours are restricted to area contained within the model grid. Contour interval 1 foot. Datum is sea level

BOUNDARY OF EVERGLADES NATIONAL PARK

ROAD

LEVEE AND ADJACENT CANAL

CANAL

Figure 32. The water-table altitude and the area of inundation in the study area at selected times, as depicted by the calibrated model for water years 1945-52.

Regional flow patterns in the Biscayne aquifer at selected times are illustrated in figure 33. The figures show arrow-tipped lines whose length and direction reflect the magnitude and direction of flow at the location represented by the base of the arrow. The scale is such that an arrow 4,000 meters long at the map scale represents a Darcian velocity of 1 ft/d or a pore velocity of 5 ft/d. The map for May 30, 1945, shows flows converging from the north and from the eastern and southern coast toward the region where water levels are below sea level.

Some of the highest water-table altitudes ever measured in the modeled region occurred in October 1947 and September and October 1948. Heavy rainfall in the summer of 1947 was followed by hurricanes on September 17 and October 17, the latter causing substantial additional rainfall. The resulting flooding in Dade County was of historic proportion and led to the formation of the Central and Southern Florida Flood Control District and to plans for construction of drainage and impoundment works to prevent a recurrence. Hurricanes on September 22 and October 5, 1948, caused substantial rainfall and flooding. Smaller peaks in the stage and water-level hydrographs in September 1945 and October 1950 were also caused by rainfall from hurricanes.

Figure 32 shows computed water-table contours and the estimated area of inundation at the height of the 1947 flood. A water-table mound is situated in the northeastern part of the area, extending eastward almost to U.S. Highway 1. The highest water-table altitudes or surface-water stages occur in the vicinity of Krome Avenue. The part of this water-table mound east of Krome Avenue corresponds closely to the area of reduced estimated hydraulic conductivity shown in figure 30. Lower aquifer transmissivity and the consequent reduced drainage in this area could have contributed to the mounding of ground water after periods of intense rainfall.

The areas shown not to be inundated during the flood include a narrow central part of the coastal ridge and a few small areas of higher elevation in Everglades National Park. The part of the coastal ridge depicted as inundated, with heads in excess of 10 ft, should be somewhat smaller because generally the model representation of land surface did not delineate areas of greater than 10-ft elevation. Figure 33 shows flow directions during this high-water period. Large flows occurred from the area of the highest water table toward the north, east, south, and southwest. Appreciable flows to the west did not occur because of low aquifer permeability in that direction.

Figure 32 shows water-table contours and areas of inundation for four times in 1950 and 1951 considered to represent typical wet-season maximum and dry-season minimum conditions, typical early wet-season conditions (when heads are rapidly increasing), and typical mid-recession conditions. The depicted typical wet-season maximum conditions (October 30, 1950) show a water-table mound in the center of the modeled area, south of Tamiami Trail, centered approximately at well G-596. The mound is lower and located westward of the simulated high-water mound of the 1947 flood. The depicted typical dry-season minimum conditions (May 30, 1951) show a small area in Everglades National Park where the water table is below sea level. In a larger surrounding area, heads are below 1 ft, the head values that characterizes areas remaining inundated near the bay coast boundaries. The north-central area of wet-season high water and the drying of the inland Everglades during dry seasons are phenomena that were described by contemporary investigators of the annual regional flow regime (Leach and others, 1972, p. 58-62).

The simulated water-table contours shown for the early wet season (August 15, 1950) have a pattern similar to that of the dry-season contours, though the heads are higher. Water-table altitudes simulated for the mid-recession period (January 15, 1950) have high gradients in the east-central coastal ridge area that are similar to those shown for the wet season, though the heads are lower. This again indicates a delayed response of aquifer drainage to wet-season recharge. The simulated flow vectors (fig. 33) also support this interpretation. Those for the typical wet-season date show that the strongest flows are in the vicinity of U.S. 1 and Ingraham Highway (fig. 16). During the earlier recession period (January 15, 1950), the most rapid flows were depicted farther inland, west of Krome Avenue. During the subsequent typical dry season, flows are negligible near U.S. 1 and Ingraham Highway, but are higher in the north-central part of the modeled region in the vicinity of G-596 and the flowing well at Grossman Hammock (fig. 16). This latter pattern remains the same during the early wet-season buildup.

A consideration of flow vectors in the wet and dry seasons (fig 33) shows that flow in the aquifer near the coasts of Biscayne and Florida Bays (fig. 16)

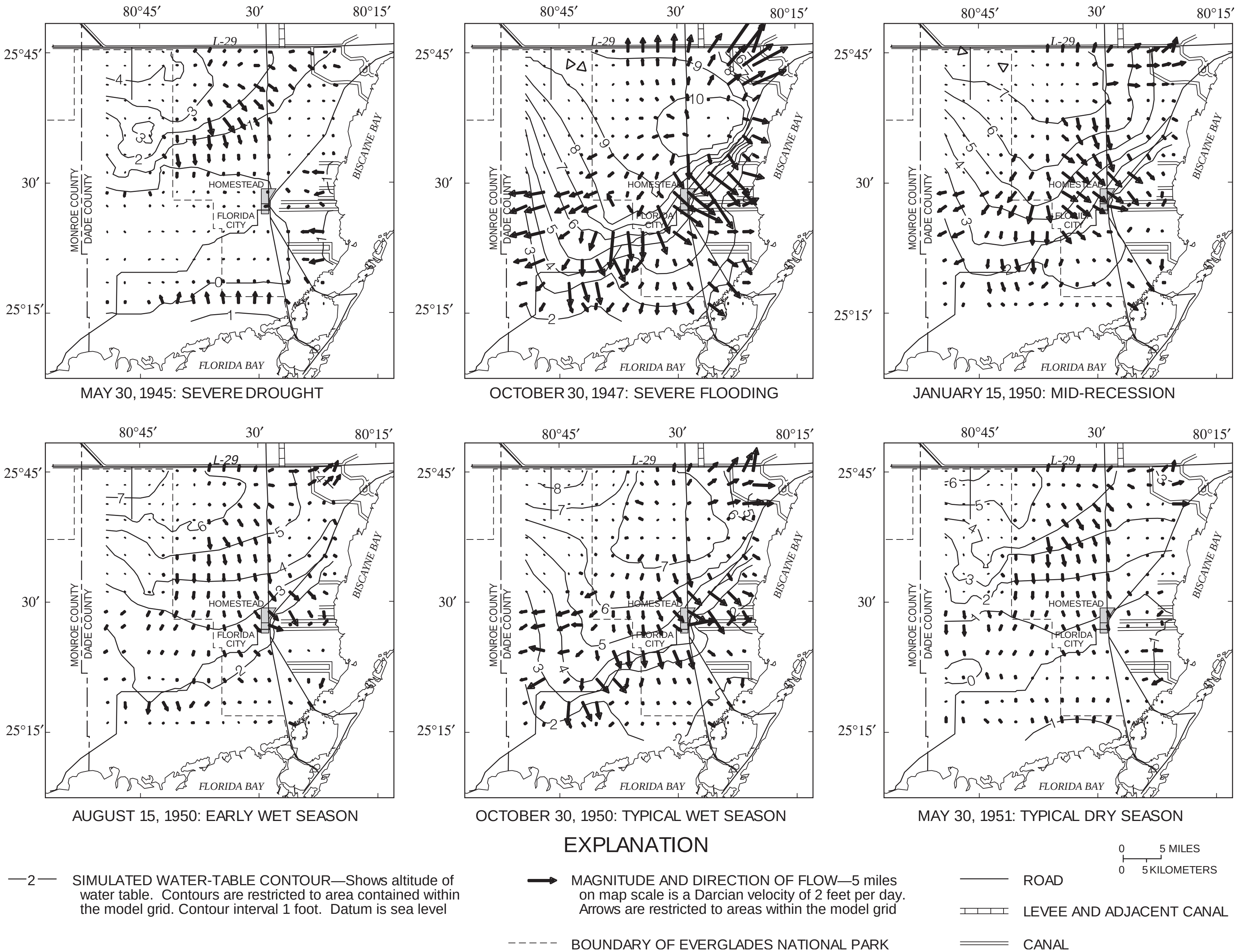


Figure 33. The magnitude and direction of flow in the Biscayne aquifer in the study area at selected times, as depicted by the calibrated model for water years 1945-52.

tends to reverse direction seasonally. A detailed examination of model output shows that this occurs in both of the model layers (3 and 4) representing the Biscayne aquifer. In a normal wet season (October 30, 1950), the rate of flow in the aquifer toward the bays decreases as the water approaches those boundaries. Significant discharges most likely occur as upward seepage into the overland flow layer, which has a substantially higher equivalent hydraulic conductivity.

Model-computed fluxes across the boundary are not correct because of the abrupt and artificial discontinuity in density that occurs there. A detailed examination of the flow regime near the saltwater boundaries has not been made. Computed heads are close to measured ones at sites near the bay coasts, and the boundary representation seems to provide a satisfactory simulation of flows farther inland.

Water Budget

A summary of regional water-budget estimates from each of the calibrated time-period models is provided in table 12. The volumes of canal discharge at the coast, rainfall recharge, evapotranspiration, and net-boundary flux other than coastal canal discharge in time period 1 (water years 1945-52) are obtained by differencing the totals at the end of the simulation time period with the totals at the end of the first year of calculations that are used to establish realistic initial conditions for the beginning of the real-time simulation.

The canal discharge volumes are computed from rates entered as input specifications to the model and represented in the model as wells in canal grid cells adjacent to the boundaries. Model-computed values in pounds have been converted to cubic feet using a conversion factor of 62.4 lb/ft^3, the average density assigned by the model to water in the aquifer.

The fluid mass balances at the end of each time-period simulation are also included in the table. The fluid mass balance is computed as:

$$\begin{array}{ll} (\Delta S+I)/E & \text{if } E > I \\ (E-\Delta S)/I & \text{if } I > E \end{array} \qquad (7)$$

where:

ΔS is the initial mass of stored fluid minus the current mass of stored fluid,

I is the incremental sum of fluid influxes, and

E is the incremental sum of fluid effluxes.

The fluid mass balance should maintain a value near 1.0 during the simulation. Substantial deviations from the value of 1.0 indicate error in the model design or application. Minor deviations from a value of 1.0 are typical and result from inherent approximation inaccuracies.

Also included in table 12 is the total mass of fluid stored in the system (total water in place) at the end of each time period. This value is included for comparison with the mass flux totals to provide insight into the significance of the water-exchange processes that occur. Because each time period ends in mid-October, these are end-of-wet-season values that reflect a high water table.

Table 12. Regional water budget for each water-management time period calibration

[Values are in cubic feet, unless otherwise noted. Positive sign indicates gain of water volume; negative sign indicates loss of water from system]

Description	Time period, in water years				
	1945-52	**1953-61**	**1962-67**	**1968-82**	**1983-89**
Rainfall recharge	1.291×10^{12}	1.382×10^{12}	0.884×10^{12}	2.346×10^{12}	1.028×10^{12}
Evapotranspiration	-1.140×10^{12}	-1.299×10^{12}	-0.825×10^{12}	-2.124×10^{12}	-1.007×10^{12}
Evapotranspiration as a percent of rainfall recharge	88.3	94.0	93.3	90.6	97.9
Net boundary flux (not including canal discharges at coast)[1]	-0.138×10^{12}	-0.063×10^{12}	$+0.050 \times 10^{12}$	$+0.031 \times 10^{12}$	$+0.075 \times 10^{12}$
Coastal canal discharges[2]	-0.049×10^{12}	-0.055×10^{12}	-0.056×10^{12}	-0.225×10^{12}	-0.119×10^{12}
Fluid mass balance at end of time period	0.9788	0.9890	0.9488	0.9927	1.0065
Total water in place at end of time period	0.369×10^{12}	0.349×10^{12}	0.354×10^{12}	0.365×10^{12}	0.348×10^{12}

[1]Includes uncontrolled discharge from simulated agricultural canals (North Canal and Florida City Canal) between 1945 and 1967. Also includes S-197 discharges after 1967.

[2]Only includes jet pumping from North Canal and Florida City Canal between 1945 and 1961. After 1961, also includes Black Creek Canal discharges. Only includes controlled east coast canal discharges (not C-111 discharges through structure S-197) between 1968 and 1989.

In the first water-management time period, rainfall recharge and evapotranspiration totals were 1.291×10^{12} and -1.140×10^{12} ft^3 (cubic feet), respectively, so the computed ratio of evapotranspiration to recharge over the 8-year period was 88.3 percent. The net-boundary flux, including simulated pumping of the agricultural canals, was -0.187×10^{12} ft^3, the negative sign indicating a net outflow across the boundaries. The water mass balance computed by the model at the end of the time period was 0.9788, which is interpreted as indicating that the flux mass influxes and effluxes are in approximate balance during the time period. The simulated “total water in place,” the volume of water contained in the aquifer, the canals, and the overland flow layer, was 0.369×10^{12} ft^3 in October 1952, about 29 percent of the total rainfall recharge for the 7-year time period, about 32 percent of estimated total evapotranspiration, and about twice the volume of net boundary flux for the time period. This demonstrates that substantial exchanges of water with the atmosphere and across lateral boundaries characterize this hydrologic system.

Additional recharge to the Biscayne aquifer occurred in the form of Floridan aquifer discharge through the flowing artesian well at Grossman Hammock. The discharge rate probably was 1,400 gal/min (gallons per minute). During the 1945-52 time period, about 8×10^{8} ft^3 of water would have been recharged at this rate, a negligible quantity compared to the principal elements of the water budget. When this rate of recharge was simulated as an injection well at the model location of the Grossman well, the effect on the local water table was negligible.

Evaluation of the Simulation for Time Period 2 (Water Years 1953-61)

Measured heads and heads computed by the calibrated model for water years 1953-61 (time period 2) are compared on plate 2. Comparison data for the entire time period were available from eight measurement locations. Nine other stations provided partial record. Nine substitutions were made for Homestead Agricultural Experiment Station monthly rainfall totals (table 10). In every case, the substituted value was from Royal Palm Ranger Station and was less than the value recorded at the Homestead Agricultural Experiment Station. Using the corrected rainfall data, a good match of observed heads was obtained at most locations in the region, including well S-196A at the Homestead rainfall station, where computed water levels at the well location were too high when only Homestead rainfall values were used. Head deviations of more than 1 ft occur for short periods of time at some stations, possibly because of the monthly averaging of rainfall data and other input parameters, and possibly also because of local variations in rainfall rates. Early data from G-757A, G-789, and NP-44 are not as well matched as later data from these locations.

From a qualitative standpoint, the poorest match of heads occurs at stations (G-620, P-33, and P-37) in Shark River Slough and the southern glades when the heads decline below land surface. Because the low hydraulic conductivity of the peat or marl layer prevents rapid drainage, the principal process determining the rate of recession in the absence of significant rainfall is evapotranspiration. Despite the use of a shallow (5 ft) extinction depth in this region, the recession curves occasionally differ appreciably from the measured data, especially when simulated heads drop below land surface earlier than the measured data, as in February 1956 at station P-33. One possible explanation is a discrepancy between rainfall amounts in the upper Everglades and those measured at the stations near Homestead and Royal Palm. An examination of the hydrograph of P-33 suggests that specified rainfall rates from the summer of 1953 to the summer of 1958 might have been too low for this area. The limited ability of the Darcian flow approximation to accurately represent surface-water flow might partly explain why simulated heads drop below land surface at times different than measured.

Wet- and Dry-Season Comparisons

The peak water table of early October 1953 (pl. 2) was caused by the passage of two tropical storms through the area, and the peak water table of June 1959 was caused by a single tropical storm that crossed the Florida Peninsula. The twin peaks of September 1960 were caused by rainfall from Hurricane Donna on September 10 and Tropical Storm Florence on September 23. The flooding from Hurricane Donna was of historic proportion, though heads depicted by the model (fig. 34) were not as high as those computed for the 1947 flood (fig. 32). As in 1947, a high-water mound is simulated in the northeastern part of the region between Krome Avenue and U.S. 1 (fig. 17). The western extent of this mound may have been reduced by northward drainage by the L-31N borrow canal, constructed in 1952.

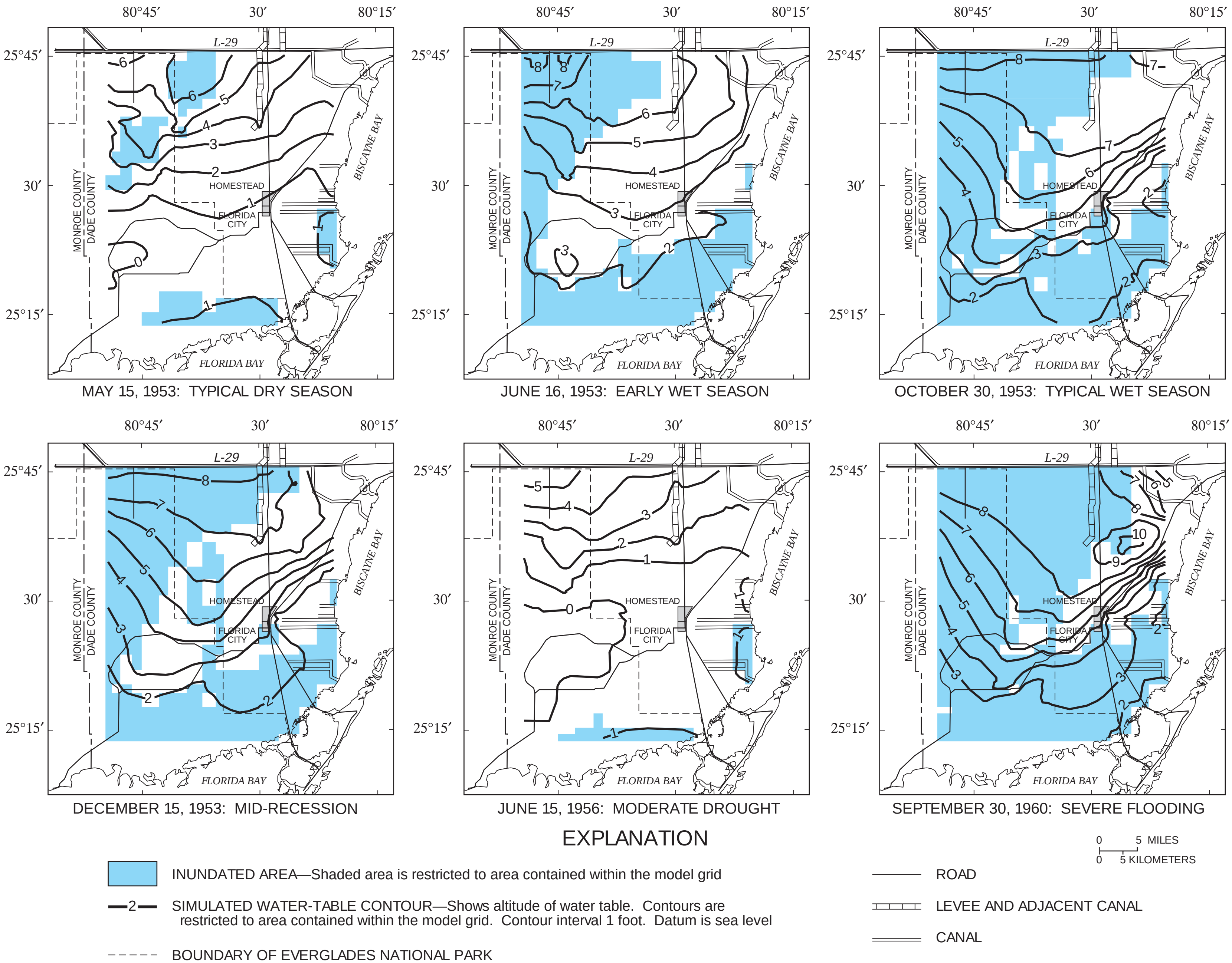


Figure 34. Map showing the water-table altitude and the area of inundation in the study area at selected times as depicted by the calibrated model for water years 1953-61.

The lowest simulated heads during water years 1953-61 (fig. 34) occurred during the dry season of 1956 and are not as low as those depicted for the 1945 dry season (fig. 32), although the general pattern, featuring a low in central and southern Shark River Slough, is similar. Figure 34 also shows computed water-table contours for selected times during the 1953 water year that represent early wet-season, typical wet-season maximum, mid-recession, and typical dry-season minimum conditions. Despite the existence of L-31N and its borrow canal, not present in the earlier time period, head distributions are qualitatively similar.

Influence of Agricultural Drainage Canals

Even by 1945, southern Dade County water-table altitudes had been altered by the local construction of canals for drainage. Water-table contour maps for the first two water-management time periods (figs. 32 and 34) depict the simulated local influence of two agricultural drainage canals, North and Florida City Canals (figs. 16 and 17), as a bending inland of head contours that cross them. These small indentations are displaced slightly with respect to the illustrated locations of the canals because the simulated canal locations were slightly displaced for convenience of model design. The location of the canals within the model grid is shown in figure 35 as a pair of solid lines in the eastern parts of rows 22 and 24. The convergence of large flow vectors in the vicinity of the canals and the flux analysis for G-612 (table 11) also demonstrate the importance of these uncontrolled canals in draining a large part of the region in the model.

Also of significance is the extensive network of secondary canals feeding into the main canals. The region drained by the secondary canals is represented in the model by a section of layer 2 in which grid cells are assigned a hydraulic conductivity of 1,000,000 ft/d (fig. 30). The model also simulates coastal pumping from each of the two main canals in October (2.5×10^7 ft^3/d [cubic feet per day]), November (5×10^7 ft^3/d), and December (2.5×10^7 ft^3/d). The assigned values are arbitrary in that no record exists of the actual rates of pumping, which was for the purpose of lowering the water table quickly after the end of wet-season rains in October.

To quantitatively assess the regional effect of the early drainage works and to illustrate how the behavior of the model is changed when canals with uncontrolled coastal drainage are represented, several sensitivity analyses were made. Results of these analyses are discussed in the following pages.

Results of First Sensitivity Analysis

In the first sensitivity analysis, the October to December pumping was deleted from the simulation. Results are shown (pl. 2) in a 1953-61 time-period hydrograph for G-612 located in the string of cells representing the North Canal. Only a minor effect on the water-table altitude, an increase in head of about 0.2 ft, is simulated during the months of the deleted pumping. The result indicates either that the pumping was ineffective in further lowering the water table, or that actual pumping rates might have been higher.

A defect in the model representation also partly explains the result of this analysis. Coastal fluxes computed by the model in the two canal cell strings for October 30, 1953 (table 11) are inland at rates of 2.4×10^6 and 4.2×10^6 ft^3/d. Thus, the model responds to simulated pumping in the easternmost canal grid cells by simulating small reverse fluxes inward from the bay. Fluxes in other coastal cells are net discharges from the modeled region into the bay. In actuality, flapper gates prevented inland movement of water from the bay to minimize saltwater intrusion. In contrast to the October computations, on May 30, 1953, near the end of the preceding dry season, canal discharges (without pumping) into the bay at rates of 7.9×10^6 and 5.8×10^6 ft^3/d were simulated. The coastal discharge rates are directly proportional to the selected equivalent hydraulic conductivity of the two canal cell strings, 3×10^8 ft/d. This value is somewhat less than the hypothetical estimate (8.3×10^8 ft/d) previously made for a “typical” canal of the region using equation 6.

Results of Second Sensitivity Analysis

In the second sensitivity analysis, the high hydraulic conductivity (1,000,000 ft/d) assigned to layer 2 grid cells in the region of dense secondary canal development (fig. 30) was reduced to the value assigned to the Biscayne aquifer (30,000 ft/d), except at some southern and eastern grid cells where the low hydraulic conductivity assigned to the layer of calcitic mud (10 ft/d) was assigned. The result is shown by comparison hydrographs (pl. 2) for well G-518 in the area of dense secondary canal development represented in the model. The hydraulic conductivity of the grid cell identified with G-518 became 30,000 ft/d in the sensitivity analysis.

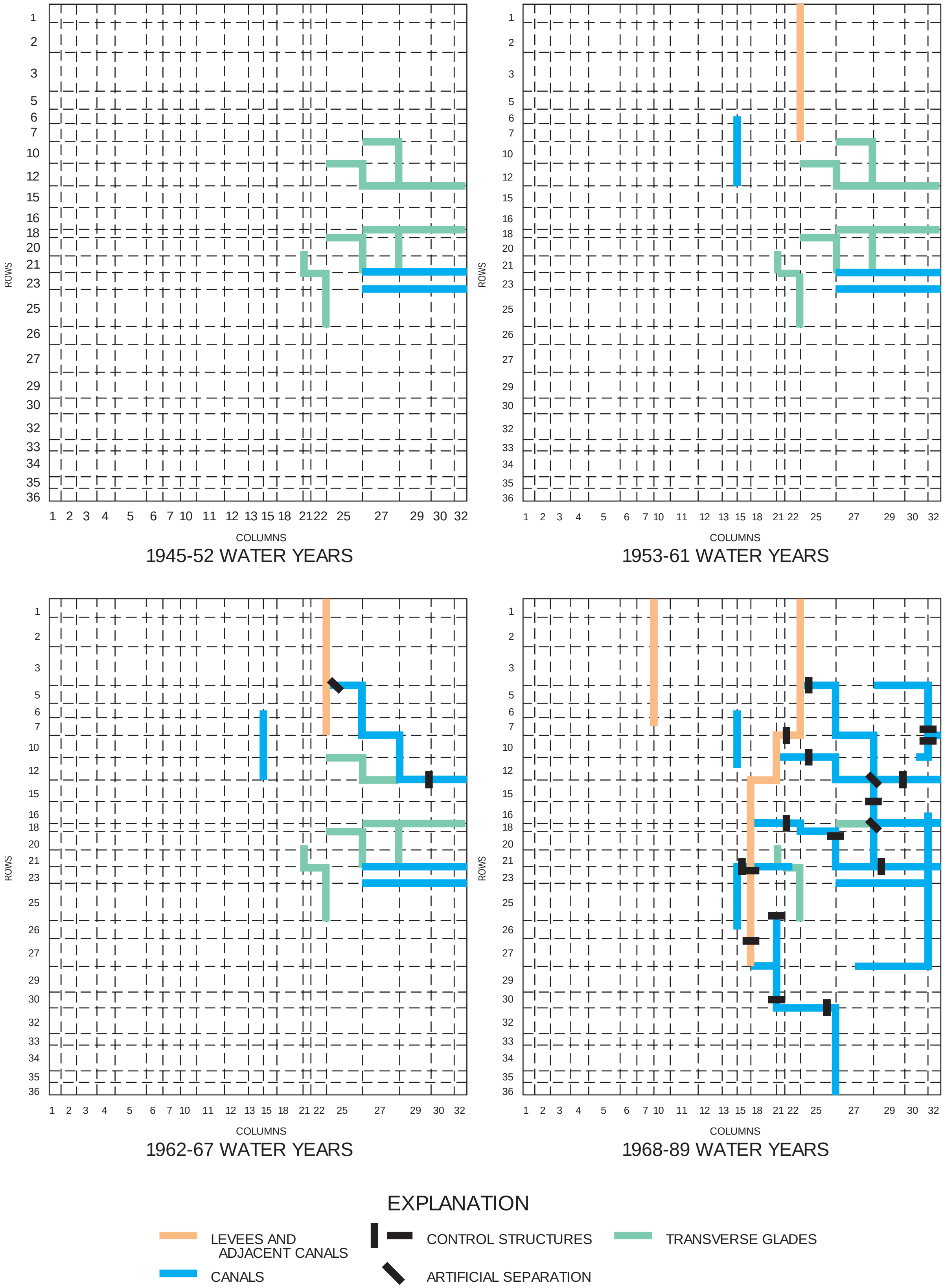


Figure 35. Model representation of transverse glades, canals, levees, and control structures in the successive water-management time periods.

Computed heads are as much as 1 ft higher during the annual summer wet seasons when the secondary canal representation is deleted. The revised hydrograph actually seems to represent measured heads better at some times, though it is too high at other times. If the result of the analysis can be considered generally indicative of the influence of the secondary canal network, the importance of these canals in lowering the October water table by as much as 1 ft in the coastal agricultural area is evident. However, this conclusion is subject to strong qualification, because it is based on the use of an arbitrary spatially averaged hydraulic conductivity coefficient (1,000,000 ft/d) to produce an acceptable match of the water level at one control point (G-518) in the affected area. The water level at the other well in the area (G-612) showed no change, being located beneath a canal cell string that retained the same high value of equivalent hydraulic conductivity.

Results of Third Sensitivity Analysis

In the third sensitivity analysis, the influence of the entire agricultural drainage system was assessed by deleting model inputs representing the main agricultural canals, the secondary canal system, and the coastal pumping. Results are illustrated (pl. 2) by comparison hydrographs from the location of well G-612 in the coastal agricultural area and well S-196A, a coastal ridge well relatively near the area of consideration (fig. 17).

At well G-612, computed water levels are as much as 2 ft higher in the wet season, rising to near land surface during 1952-57, which were relatively dry years. The only period in which computed water levels were unchanged are the final months of the relatively severe dry seasons of 1955 and 1956. Water levels at S-196A are also increased by a lesser amount, generally less than 1 ft. The water levels computed at the G-612 cell with the canal drainage system removed would not be acceptable as part of a successful calibration, and drastic unsupportable changes in aquifer parameters would be required to compensate.

The regional effect of the agricultural drainage system at selected times is shown (fig. 36) by contours of water-table altitude increases that are computed when the representation of the system is deleted from the model. Dry-season (May 15, 1953) and wet-season (October 30, 1953) increases are shown. The effect of removing the drainage system on the simulated dry-season water table extends into the rocky glades region, and heads in parts of the southern and eastern glades increase slightly. Appreciable effects in the wet season are generally restricted to noninundated parts of the region. On October 30, 1953, computed heads east of Homestead increased as much as 1.5 ft when the drainage system was not simulated. Generally, the wet-season effects are more intense, though more localized.

Heads computed without representation of the agricultural drainage system, given atmospheric conditions prevailing in 1953, probably are more representative of conditions prevailing before modern economic development of the study area than those of the calibrated model. These heads and depicted areas of inundation (fig. 37) resemble historic predevelopment conditions in southern Dade County more closely than any others simulated as part of this study. The simulation that generated this head distribution also omitted the representation of L-31N and its borrow canal. Properties of this simulation that might cause the heads to be different from ones that would have occurred before manmade water management on the east coast are described below.

- Specified pressures along the northern boundary might reflect drainage by the major east coast canals connecting Lake Okeechobee with the coast to an extent that is undetermined.
- Specified boundary conditions along the northern boundary east of Krome Avenue (fig. 17) were probably slightly affected by L-30, the Dade-Broward Levee, and drainage by Coral Gables and Snapper Creek Canals and other canals north of the study area east of Krome Avenue.
- The numerous culverts under Tamiami Trail before 1962 (fig. 10), though connected by the shallow borrow canal of the time, might not have fully mitigated its influence as an impoundment.

Influence of Levee 31N

The model was also used to assess the hydraulic influence of L-31N and its adjacent borrow canal, both constructed in 1952. The levee and canal are represented by the upper parts of columns 23 and 24 (fig. 35). Because the L-31N borrow canal was plugged below the Tamiami Canal (fig. 18), the canal cell string is closed at the boundary, so that no boundary fluxes occur at the end of the canal cell string. Figure 34 shows a bending of water-table contours as they cross the canal. The contours bend northward near the upper end of the levee and canal, indicating a lowering of heads by the canal, and southward near

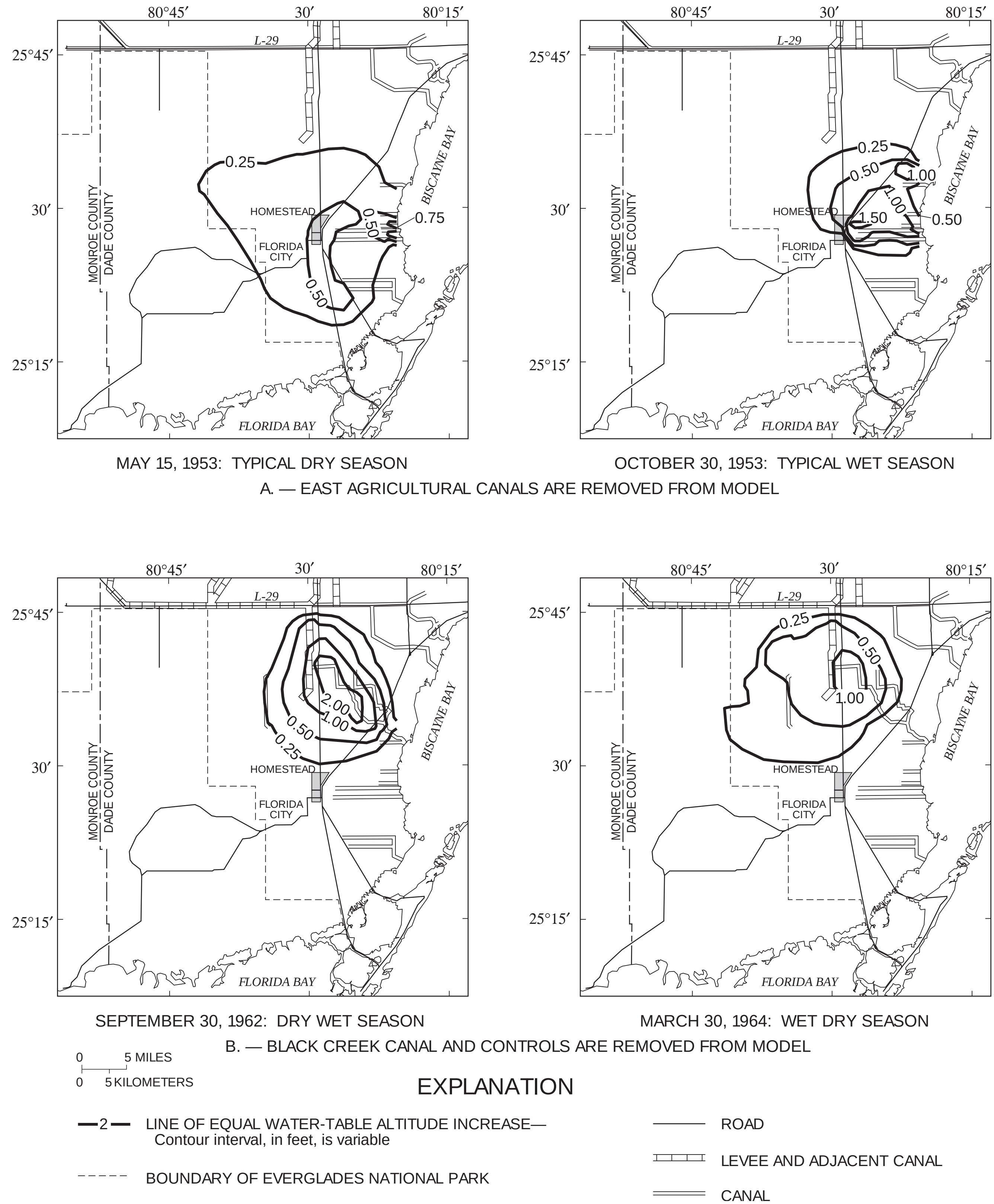


Figure 36. The rise of the computed water table at selected times when: (A) east coastal agricultural canals are removed (water years 1953-61), and (B) Black Creek Canal and controls are removed (water years 1962-67).

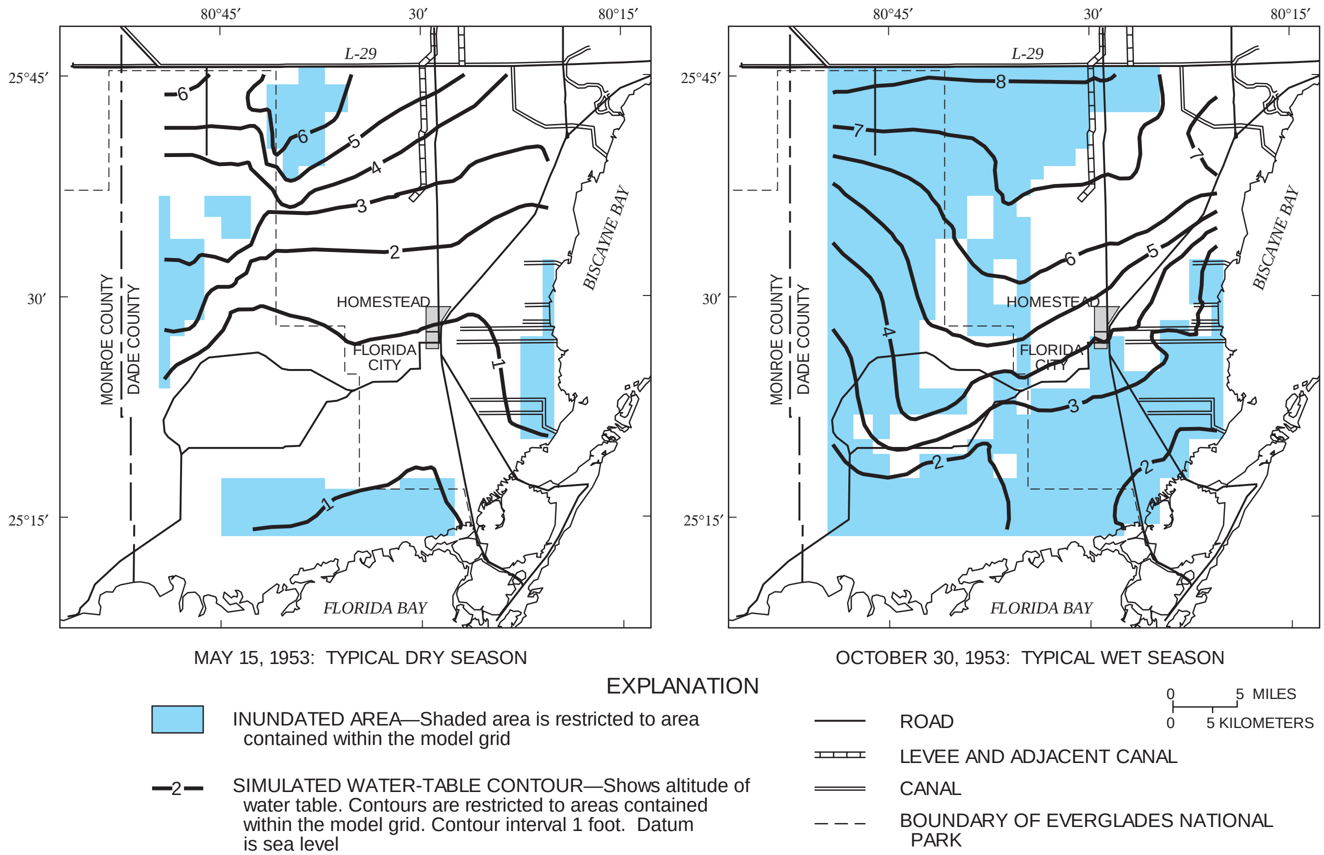


Figure 37. The high and low water tables and corresponding areas of inundation that might have occurred in the study area in 1953 if no canals had existed in the area other than the Tamiami, Coral Gables, and Snapper Creek Canals.

the lower end, indicating a raising of heads by the canal. This shows the simulated effect of the unimpeded reach of the borrow canal in serving as a "short circuit" for water circulation in the upper part of the aquifer. The simulated stage gradient in the canal reach is less than the simulated head gradient in the aquifer a moderate distance from the canal. Upstream canal stages are lower than heads in the surrounding aquifer, and downstream canal stages are higher. This simulated result depends on the assumption of an unimpeded hydraulic connection between canal and aquifer. The apparent effect on aquifer heads, however, is depicted by the model as somewhat localized.

For an additional assessment of the influence of L-31N, a sensitivity analysis was made in which L-31N and canal representations were deleted. Only at well G-596 close to the southern end of the canal and levee and at well G-757A farther southeast was there an appreciable effect on the computed water level. With the levee and canal removed, computed water levels at G-596 were lowered by as much as 0.6 ft, the greatest reduction occurring during the dry season. When water levels approached land surface, the difference was negligible. The effect on G-596 water levels is primarily the dry-season influence of the borrow canal, which raises the water table in the vicinity of the lower part of the canal reach. The distance between the G-596 grid cell and the edge of the nearest canal grid cell was 3,140 ft. The actual distance between the well and the canal is about 4,500 ft.

Transverse Glades

The representation in certain cell strings (fig. 35) of land-surface elevations 1.5 to 8.0 ft lower than surrounding land surface was to represent the transverse glades and to simulate their effect on the hydrologic regime (the same cell strings were later used to represent canals). The transverse glades leading to the coast in row 14 are the upper and lower parts of Black Creek Glade (called Peters Prairie on old maps of Dade County). The indicated string of cells leading to the coast in row 17 represents the Princeton Glade (Caldwell Prairie), and the indicated strings of cells leading downward to North Canal in row 22 represents parts of the Mowry Glade (Sherritt's Prairie in column 26 and Gossman Prairie in column 28). The indicated cells leading southward in column 24 are a transverse glade that separates Homestead and Florida City (Long Prairie).

Inspection of a matrix of computed overland flow depths corresponding to the typical high-water conditions of October 30, 1953, shows most of the transverse-glade cell strings to be dry, although the computed depth below land surface to the water table is generally less than 1 ft. Only the cell string corresponding to Black Creek Glade is shown to be inundated (flowing) from about 2 mi west of U.S. 1 to the coast, with flow depths ranging from 0.10 to 1.25 ft. An inspection of simulation results for September 30, 1960, during the flood following Hurricane Donna and Tropical Storm Florence, shows all glade cell strings except those for Princeton Glade to be inundated (flowing). Simulated depths of inundation in Black Creek Glade are as much as 4 ft.

The depths assigned to the transverse glade cell strings were carefully selected based on detailed land-surface elevation maps predating canal development. Thus, the evidence provided by the model is based on accurate model construction. Simulated results generally indicate that the glades flowed infrequently and only when exceptionally high-water conditions occurred in the historical period following the beginning of Everglades drainage in the early 1900's. The glade indicated to be the most likely to flow under typical seasonal high water conditions is Black Creek Glade. Perhaps it is not entirely coincidental that this glade extended into the area in the northeastern part of the modeled region where high-water mounding regularly occurred in periods of heavy rainfall (figs. 32 and 34). Model results are consistent with the memories of some long-time residents of the area, who claim that at least some of the transverse glades typically had flowing water in the summer months. However, quantitative stage or flow data to validate these claims or the model results are lacking.

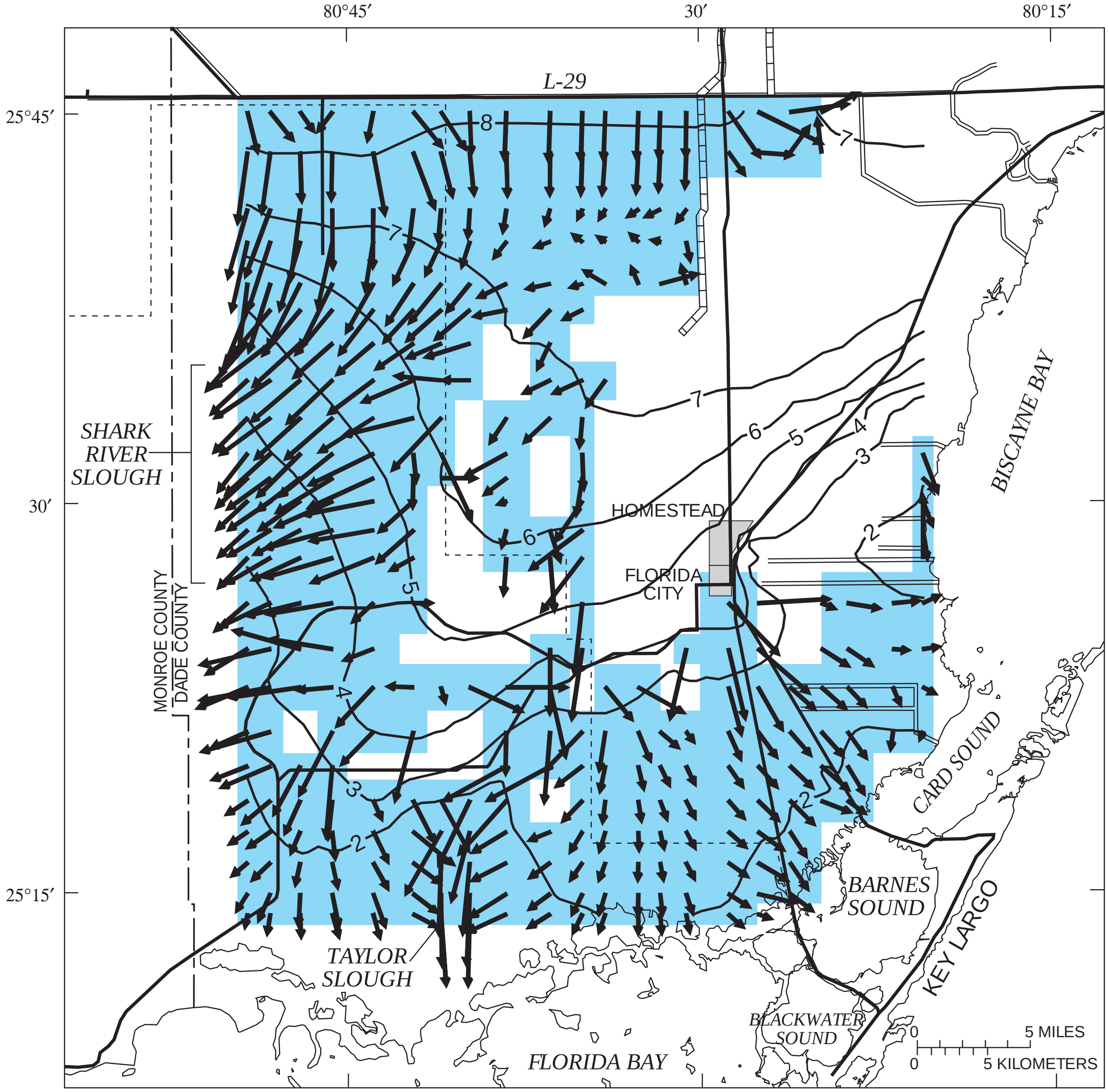
Surface Flows

As previously noted, one of the principal objectives of model development was to replicate stages in the surface flow region with sufficient accuracy that their influence on ground-water flow in the Biscayne aquifer would be adequately represented. A comparison of computed and observed heads at stations P-33, P-37, G-620, G-596, NP-44, and NP-46 (pl. 2) has helped to verify the simulation of surface-water stages from 1950 to 1961. Accurate simulation of flow in the surface-water body was not a simulation objective, and in fact, would be unverifiable given the lack of flow velocity data from the inundated region. However, an examination of simulated surface flows is useful in illustrating how the revised SWIP code works, and a qualitative comparison of simulated and observed surface flow patterns serves to lend credence to the assumptions made and the adequacy of the model to accomplish its principal purpose.

Figure 38 shows the simulated directions and magnitudes of surface flows during an average wet season (October 30, 1953). Surface flows in Black Creek Glade were omitted from the figure for clarity. The scaling is such that a vector 5 mi long at the map scale represents a flow velocity of 400 ft/d. Many of the illustrated flow vectors in Shark River Slough represent computed velocities as great as 280 ft/d. The direction of flows in the slough is approximately correct, as verified by a comparison with orientation of tree islands shown in figures 8 and 9.

The area of inundation shown in figure 38 generally corresponds to the area indicated in figure 8 as being subject to wet-season inundation, although a few differences are indicated in the central rocky glades region and between the two park roads. The estimated region of inundation shown in figure 8 was based on widely scattered data from a more recent time period in which water-management construction has somewhat modified the distribution of surface flows. The simulated area of inundation shown in figure 38 is sufficiently widespread that overland flow (layer 1) grid cells representing the shallow, muck-bottom channels in the rocky glades region leading southward toward Taylor Slough (fig. 3) were saturated (simulated as containing flowing surface water). The Taylor Slough representation acted as a drain for the simulated surface-water flow system. The convergence of flow in lower Taylor Slough is indicated by the longer vectors in that region, where the equivalent hydraulic conductivity was 1×10^8 ft/d (fig. 30) and computed flow rates were as high as 2,675 ft/d. The flow vectors in this region have been substantially reduced in scale to clarify the illustration.

In most of the remaining southern and eastern glades, computed surface flows are less rapid than in the sloughs. Computed surface flow velocities in this area are as high as 230 ft/d and decrease southward to as little as 13 ft/d at the boundary representing the



EXPLANATION

OCTOBER 30, 1953: SURFACE-WATER FLOWS

INUNDATED AREA—Shaded area is restricted to area contained within the model grid

—2— SIMULATED WATER-TABLE CONTOUR—Shows altitude of water table. Contours are restricted to area contained within the model grid. Contour interval 1 foot. Datum is sea level

MAGNITUDE AND DIRECTION OF FLOW—5 miles on map scale is a velocity of 400 feet per day. The scale of the vectors in lower Taylor Slough has been reduced for clarity. Arrows are restricted to area contained within the model grid

ROAD

LEVEE AND ADJACENT CANAL

CANAL

BOUNDARY OF EVERGLADES NATIONAL PARK

Figure 38. Model depiction of the area of inundation on October 30, 1953, with vectors showing the computed magnitude and direction of surface-water flow in the inundated region.

coast of Florida Bay. The horizontal ground-water pore velocity is as high as 10 ft/d south of Florida City and also decreases southward. Near the coast of Florida Bay, the direction of simulated flow in the Biscayne aquifer is northerly rather than southerly because of the influence of the pressure based on sea-water density assigned to the boundary. This suggests that a freshwater-saltwater interface occurs north of the coastline, as has been suggested by other studies (Meyer and Hull, 1969; Sonntag, 1987). The north-south location south of which northerly flows are simulated varies seasonally, suggesting that the saltwater interface position may change seasonally.

Water Budget

The regional water-budget estimates for the second water-management time period, water years 1953-61 (table 12), indicate that the ratio of evapotranspiration (-1.299 x 10^{12} ft^3) to rainfall recharge (1.382 x 10^{12} ft^3) was 94.0 percent, slightly higher than in the first time period (water years 1945-52). The net boundary flux (-0.063 x 10^{12} ft^3), including simulated pumping of the agricultural canals, was smaller and a net loss from the system. The net boundary flux is larger, however, than the net difference (0.083 x 10^{12} ft^3) between rainfall recharge and evapotranspiration. The computed water mass balance at the end of the time period was 0.9890, indicating that computed influxes and effluxes of water were in approximate balance. The simulated “total water in place” was 0.349 x 10^{12} ft^3 in October 1961.

Evaluation of the Simulation for Time Period 3 (Water Years 1962-67)

Measured heads and heads computed by the calibrated model for water years 1962-67 (time period 3) are compared on plate 3. A total of 23 sites (fig. 19) provided comparison data for all or nearly all of the time period, and one other (G-1251) provided 2 1/2 years of data. Three substitutions were made for Homestead Agricultural Experiment Station monthly rainfall totals (table 10). In each case, the substituted value was greater than the Homestead value. In two cases, the substituted values were from Royal Palm Ranger Station; the third was from Miami International Airport.

At stations (S-196A and F-358) near the source of most of the rainfall data, computed water levels match measured ones quite well throughout the time period. At many stations distant from the rainfall data source, lack of agreement during some months is probably related to local differences in rainfall amounts, but agreement with the general trend of the measured data is good. Computed and observed data do not match well at the location of well S-182A near Belaire Canal (C-1N), completed in 1962. The rapid aquifer drainage by the nearby canal, readily apparent from the greatly changed pattern of seasonal water-level variations compared with that of earlier time periods, is not simulated well (possibly because of the monthly average nature of model inputs). Comparing computed and measured heads at stations in Shark River Slough and Long Pine Key, it seems that these latter regions may have received substantially more rainfall in June 1965 than recorded at Royal Palm Ranger Station.

Wet- and Dry-Season Comparisons

Despite the occurrences of hurricanes or tropical storms in 1962, 1964, 1965, and 1966, only Hurricane Betsy on September 8, 1965, delivered a significant amount of rainfall. This hurricane also caused a 9.8-ft storm surge from the lower part of Biscayne Bay (Hull and Meyer, 1973). Other wet-season peak water tables, including the high for the time period in September 1963, were the result of unusually large amounts of rainfall from summer and autumn storms. The 1962-67 time period was marked by pronounced dry seasons, with those of 1962, 1963, 1965, and 1967 lowering heads below sea level over large parts of the modeled area. Water-table contours and areas of inundation based on the computations for the September 1963 water-table peak and the previous 1963 spring low are shown in figure 39. Also shown are water-table contours for dry wet season (lower-than-average yearly high-water peak) conditions (September 30, 1962), mid-recession period conditions (December 15, 1962), early wet-season conditions (June 15, 1963), and wet-dry season (higher-than-normal yearly low) conditions (March 30, 1964).

The dry-period water-table contours of April 30, 1963, seem to be simulated accurately based on head comparisons at various stations (pl. 3), and figure 39 shows a water table as low as in early summer 1945. A region where heads are more than 1 ft below land surface is shown to occur in lower Shark River Slough. Heads this low were not simulated for the 1945 period of low water, though the 1945-52 simulation did not quite achieve the measured lows of that year. The

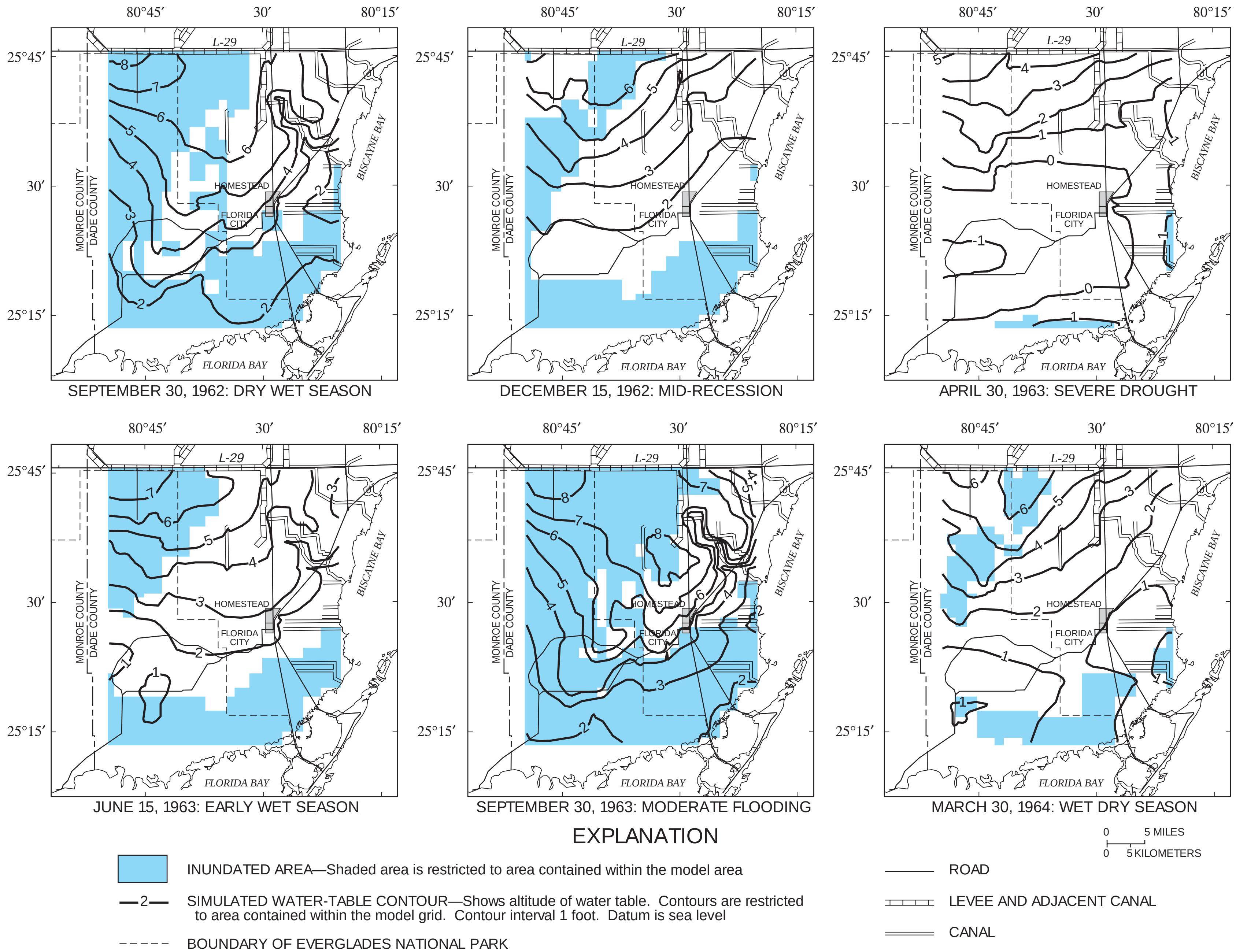


Figure 39. The water-table altitude and the area of inundation in the study area at selected times, as depicted by the calibrated model for water years 1962-67.

high-water peak of September 30, 1963, also simulated well at most data sites, is depicted by the model as having caused less widespread flooding than occurred in 1947 or after Hurricane Donna in 1960. The recorded rainfall for the month was equal to or greater than that of the months of each of the other events but was preceded by a drier summer.

Influence of Black Creek Canal

Another reason for the limited flooding during the unusually high water conditions of September 1963 (fig. 39) was the presence of Black Creek Canal, constructed through the center of the area of highest water levels in the 1947 and 1960 floods (figs. 32 and 34). A smaller water-level mound remains south and west of the southern end of L-31N. The steep head gradients simulated near Black Creek Canal during this and other high-water periods are an indication of the effectiveness of the canal as a local drain when control structures are opened. The contours are slightly displaced relative to the location of Black Creek Canal indicated in the figure because model design limitations required a generalization of the meandering course of the canal.

The location of Black Creek Canal in the model grid is shown in figure 35 as a zigzag string of canal cells that begins adjacent to L-31N in row 4 and reaches the eastern boundary in row 13. The canal is separated from the L-31N borrow canal by a simulated control structure, which is given a control elevation high enough to forestall simulated openings. No such structure actually exists at this location, but the artifice of simulating a fictitious structure was used to facilitate accurate model construction. The stage-divide structure near U.S. 1 is represented and given a control elevation of 5.5 ft. The canal cell string is blocked at the coastal boundary, and discharges are represented by pumping wells as previously described. Because discharge was not measured before 1969, long-term monthly averages based on more recent data were used in this time period. The equivalent hydraulic conductivity assigned to the canal cells was 3×10^8 ft/d. Examination of computed heads shows that the model simulates a stage difference of 1 ft on March 30, 1964, between the upstream end of the canal, adjacent to L-31N, and S-148, which was closed on that date. This stage loss in the 10.3-mi canal reach generally is comparable to the 0.3 ft measured later in the 1984-86 water years (fig. 24). The Belaire Canal (C-1N), draining into Black Creek Canal from the north, was not explicitly represented in the model grid.

To assess the regional extent of the influence of Black Creek Canal on the water table, a sensitivity analysis was made in which the representations of the canal, control structures, and coastal discharge were deleted. The resulting simulated water levels at the nodal locations of G-855, G-596, G-863, and G-614 are compared on plate 3 with measured water levels and water levels from the calibrated model. G-855 and G-596 are near the upper end of the canal and should show the effect of canal drainage most of the year, and especially during wet-season peaks when the control structures were normally open. G-855 is 7,500 ft from the canal in the model grid, but the actual distance is 6,500 ft. G-596 is 10,175 ft from the head of the canal in the model grid, but the actual distance is 8,250 ft.

At G-855, simulated wet-season peak water levels are as much as 1.5 ft higher when the canal is not represented. During the dry-season lows of 1963 and 1965, simulated water levels were still 0.2 ft higher with the canal removed. At G-596, the effect was only slightly less pronounced. In the model design, the cell representing G-614 is relatively near (8,359 ft) the canal because of the generalization of the canal representation, but the well is actually more distant (17,000 ft). When the canal is removed, G-614 water levels are simulated to be as much as 0.6 ft higher in the wet season but are virtually the same as those of the calibrated model during the dry season. Well G-863 in the rocky glades south of the flowing Grossman well is about 43,000 ft from the head of the canal, and only a slight effect (less than 0.5 ft higher when the canal was removed) was evident during the recession period following wet-season highs. The effect probably lags the local wet-season peak drainage of the canal because of the large intervening distance.

Other depictions of the regional influence of Black Creek Canal are the contour maps of the head increases computed when the Black Creek Canal representation is deleted (fig. 36). These are presented for a day during a typical wet-season high-water period (September 30, 1962) when control structures were open and for a day during a typical dry-season low-water period (March 30, 1964). In the high-water period, increases of as much as 2 ft are simulated in the northeastern part of the study area, indicating water levels as high as occurred in earlier times before the construction of the canal. The hydraulic influence of the canal apparently is negligible in the inundated region west of the central rocky glades region and is also negligible south of Homestead. In the dry period, the influence of the canal drainage apparently extends westward to the boundary of Everglades National Park, the eastern limit of flowing surface water, and southward to where flowing surface water occurred in the southern glades. The effects are not as pronounced as in the wet season, though distributed over a larger area. The effect on dry-season heads is partly a residual effect from the use of the canal for drainage during the previous wet season.

Black Creek Canal is indicated by the model analysis to have substantially altered the natural patterns of drainage in the modeled area, particularly during high water and recession periods (fig. 40). Compared to the flood, average high water, and recession flow regimes of 1947 and 1950 (fig. 33), northward flows in the Biscayne aquifer toward Tamiami Trail have been reduced, and eastward flows in the vicinity of Black Creek Canal have been increased by the drainage of the high-water area shown in figure 33. In the recession period, strong aquifer drainage now occurs west of the L-31N segment. In the mild and severe dry periods, aquifer drainage in the upper part of the Black Creek Canal basin is greatly enhanced.

Influence of Levee 29

One example of canal and levee construction for the purpose of water management that had a significant influence on aquifer flows in the modeled area in the 1962-67 time period could not be represented by changes in the grid design of the model or by coefficient assignments because it occurred just outside of the northern boundary of the mode. This was the construction of L-29 along the Tamiami Trail to control southward flows in the Everglades. The section of the Tamiami Canal east of L-67A was blocked at L-67A (fig. 18) from the L-29 borrow canal and was connected to the section of the Tamiami Canal west of L-67A by a structure that was seldom opened. East of L-67A, spoil material for the construction of L-29 was obtained by further excavating Tamiami Canal so that it became deeper and more arterial. The eastern end of L-29 was at the northern end of L-31N where the Tamiami Canal was blocked from the section of the Tamiami Canal east of L-31N.

The effect of L-29 and the canal blockages on recorded stages in the canal section between L-67A and L-31N is not entirely clear, as evidenced by the table shown below of monthly average stages measured at Bridge 45 during time period 1 (water years 1945-52), time period 2 (water years 1953-61), time period 3 (water years 1962-67), and time period 4 (water years 1968-82). Average stages at the recorder in time periods 3 and 4 generally are about 1 ft lower than those of time period 2. This seems to be clear evidence of a reduction in average stage. However, they exceed average water levels of time period 1 in several months.

Because of the apparent data trends, a question is posed concerning how a general variation in stages used to specify heads along this part of the northern boundary of the model influences computed heads in the modeled region. A corollary question is whether heads in the modeled region would have been significantly higher if the canal blockage at L-67A had not lowered stages in the canal reach between L-67A and L-31N. A sensitivity analysis was made in which specified heads along the northern boundary east of L-67A were increased by 1 ft throughout the 1962-67 time period computation. L-31N stages were not at issue in this analysis because the canal was cut off from Tamiami Canal in this time period, and stages were mainly influenced by the local water table.

Time period	Monthly average stages, in feet above sea level											
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept
1	7.48	7.37	7.04	6.78	6.35	5.83	5.17	4.68	5.18	6.03	6.32	6.99
2	8.17	8.14	7.85	7.58	7.28	6.91	6.52	6.32	6.52	7.05	7.21	7.57
3	6.95	6.86	6.61	6.44	6.24	6.12	5.65	5.27	6.07	6.47	6.62	6.85
4	7.03	6.95	6.82	6.72	6.57	6.21	5.68	5.70	6.31	6.68	6.79	7.02

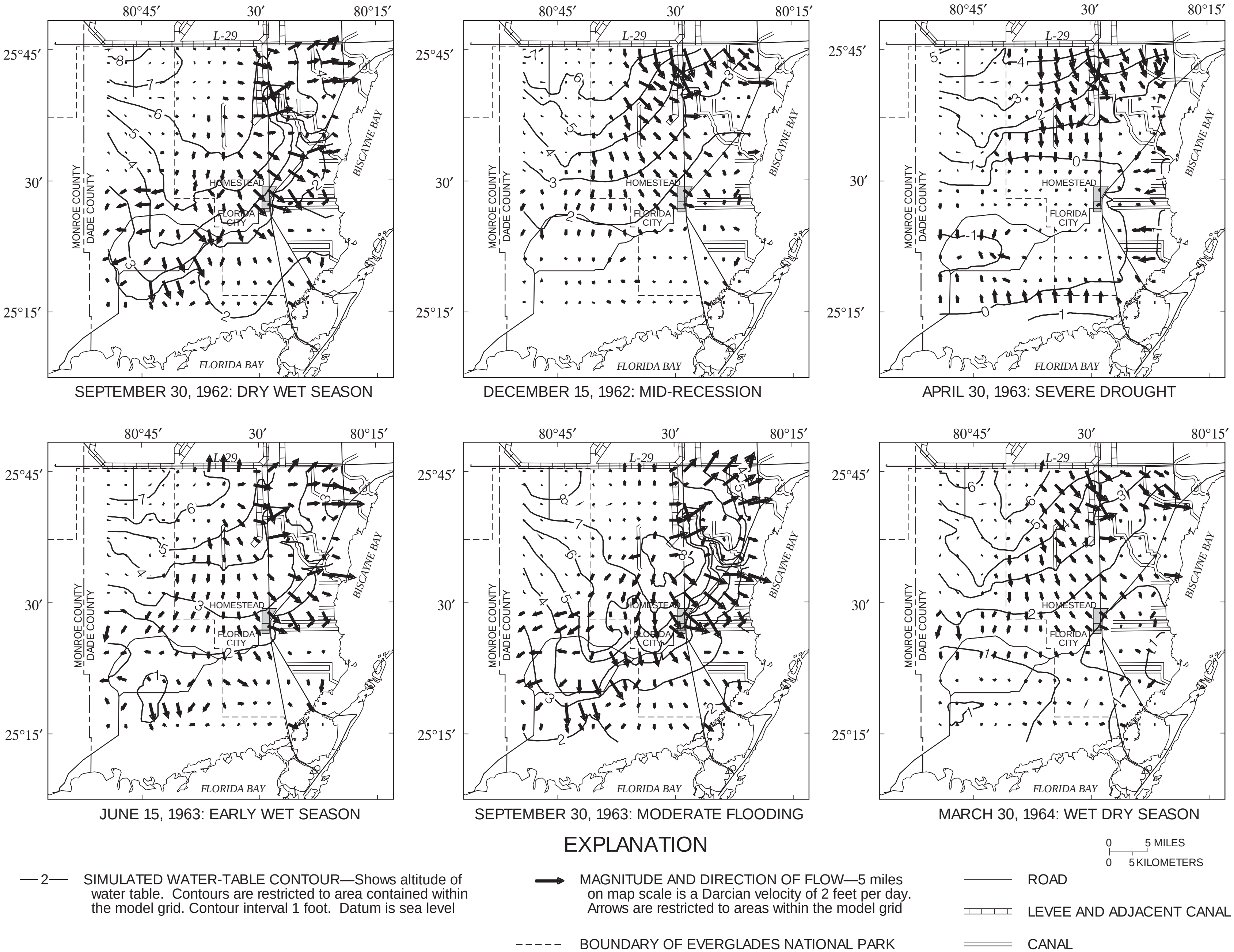


Figure 40. The magnitude and direction of flow in the Biscayne aquifer in the study area at selected times, as depicted by the calibrated model for water years 1962-67.

Results are indicated by comparison hydrographs (pl. 3) for wells G-861 and G-863, selected because they are farthest from east and west model boundaries and from the northern boundaries west of L-67A and east of L-31N. These boundary conditions remained unchanged and could constrain local head variations. Both wells are in the northern half of the modeled region (fig. 19). The change in simulated water levels at wells G-861 and G-863 is greatest (pl. 3) when the increased boundary heads cause water levels at the wells to remain above land surface for longer periods of time than in the calibrated model because surface-water stages do not decline as rapidly as water-table altitudes. When water levels remained below land surface for lengthy periods of time, as in March 1962, the computed water-level increase resulting from the boundary variation was about 0.4 ft at G-861 (about 8 mi south of the boundary) and 0.15 ft at G-863 (about 13.5 mi south of the boundary). Thus, it seems likely that blockage of overland flow by L-29 and lowering of the stage in the eastern section of the Tamiami Canal did lower the water table in the northern half of the modeled region by a small amount. The effect contributed to the influence of Black Creek Canal drainage in lowering peak water-table altitudes throughout the northern part of the study area. The illustrated variation is a small fraction of the variations induced when rainfall and evapotranspiration rates were each varied by 20 percent (pl. 1).

Water Budget

The regional water-budget estimates for the third water-management time period, water years 1962-67 (table 12), indicate that the ratio of evapotranspiration (-0.825 x 10^{12} ft^3) to rainfall recharge (0.884 x 10^{12} (ft^3) was 93.3 percent. Compared to recharge and evapotranspiration quantities, the net boundary flux (-0.006 x 10^{12} ft^3), including uncontrolled drainage by the agricultural canals, jet pumping of the agricultural canals, and the estimated drainage of Black Creek Canal, is a negligible net loss from the system. It is even small compared to the net difference (0.059 x 10^{12} ft^3) between rainfall recharge and evapotranspiration. The computed water mass balance at the end of the time period was 0.9488, less than the ideal value of 1.0000, but not enough to indicate significant error in the model algorithms. The simulated “total water in place” was 0.354 x 10^{12} ft^3 at the end of the time period in October 1967.

Evaluation of the Simulation for Time Period 4 (Water Years 1968-82)

Measured heads and heads computed by the calibrated model for water years 1968-82 (time period 4) are compared on plates 4 to 7. A total of 21 stations provided records that spanned more than two-thirds of the period, 8 stations provided records spanning most of the last 10 years, and 1 station provided 4 years of data. Ten substitutions for Homestead Agricultural Experiment Station monthly rainfall totals were made (table 10). All substituted values are from the Royal Palm Ranger Station or the Homestead Field Station of the South Florida Water Management District. Five substituted values are larger than the measured Homestead Agricultural Experiment Station values, and five are smaller. The general match of computed and measured heads is good, but simulation difficulties occur during periods of exceptionally high rainfall in the wet seasons of 1968, 1969, and 1981 that suggest an inherent limitation in the simulation capability of the model.

Representation of the Flood-Control System

The grid representation of major parts of the flood-control system completed in 1967 (fig. 20) is depicted in figure 35. L-67 Extended was represented by a thin string of cells of low hydraulic conductivity in column 9 of the upper layer. The adjacent canal west of the levee was represented as a string of canal cells in column 8 of layer 2. The Princeton (C-102) and Mowry (C-103) Canals originate at L-31N in rows 11 and 17 and reach the bay boundary in rows 17 and 24--the latter replacing representation of the North Canal in row 24. The canals are slightly displaced from their true locations for convenience of model design. Mowry Canal North (C-103N) is represented as a string of cells in column 28, cut off from the Princeton Canal cell string by a fictitious control structure with a specified control elevation higher than any possible canal stage. The Princeton Canal cell string is separated from the Black Creek Canal cell string by another fictitious control structure that never opens. Two control structures are simulated in the Princeton Canal cell string to represent the operation of S-194 and S-165. Three control structures are simulated in the Mowry Canal cell string to represent the operation of S-196, S-167, and S-179. Control elevations at the automatic gates are simulated to be as specified in table 4. Manually operated gates S-194 and S-196 were assigned control elevations of 6.0 and 5.5 ft to represent customary operational procedure.

L-31N Remainder extends downward from where the previous cell string ended in row 9 (where control structure S-173 is now simulated) as a sequence of canal-levee or canal cells (identified as C-111 below row 22) and leading to the southern boundary in column 26. Simulated control structures near rows 22, 26, 30, and in 31 near column 26 represent structures S-176, S-177, S-18C, and S-197. Specified control elevations are as in table 4, and 1.8 ft is specified for manually operated structure S-197. The canal string was uncontrolled at the coast; however, the simulated structure representing S-197 was nearly always closed.

Crossing the L-31N levee string in row 22 is a cell string that represents C-113 to the east and the L-31W canal to the west. The latter bends south in column 14. The southern east-west section of the canal was not represented nor was structure S-175. A simulated structure representing S-174 separates the L-31W canal from the L-31N canal. It was not deemed necessary to simulate L-31W because it has little influence on the surface-water system. Neither were levees along C-111 below row 27 represented, as they probably are rarely used to separate stages in the southern glades. A simulation of manually operated control-structure S-178 at row 26 headed a string of canal cells in column 20 that represented C-111E.

A string of canal cells extending south in column 31 from rows 16 to 28 and then west to column 27 represented the canal adjacent to coastal levee L-31E, and outlets to the sea were specified in columns 17 and 22, representing the Princeton and Mowry Canals. However, both strings were closed at the bay boundary, and coastal discharges were simulated by pumping wells. As Princeton Canal discharges were unknown, Mowry Canal discharges were assigned to each well. Representation of coastal levee L-31E was not deemed necessary because its purpose is to impound storm tides from Biscayne Bay, which did not occur within the part of the overall simulation time period (water years 1968-89) when the levee was present.

It was considered necessary to simulate some drainage from the northeastern part of the modeled area where a network of canals (C-1N, C-100, C-100A, C-100B, C-100C, and C-2) and many control structures exist, but simulation objectives did not warrant the effort required for a detailed representation. Therefore, a string of cells in row 4 connecting to a string in column 31 was used as a generalized depiction of area drainage. A closed outlet to the bay was provided in row 8 but was blocked from upper and lower parts of the cell string in column 31 by hypothetical control structures assigned a control elevation of 5 ft. All of the new canals were assigned equivalent hydraulic conductivities of 2 x 10^8 ft/d.

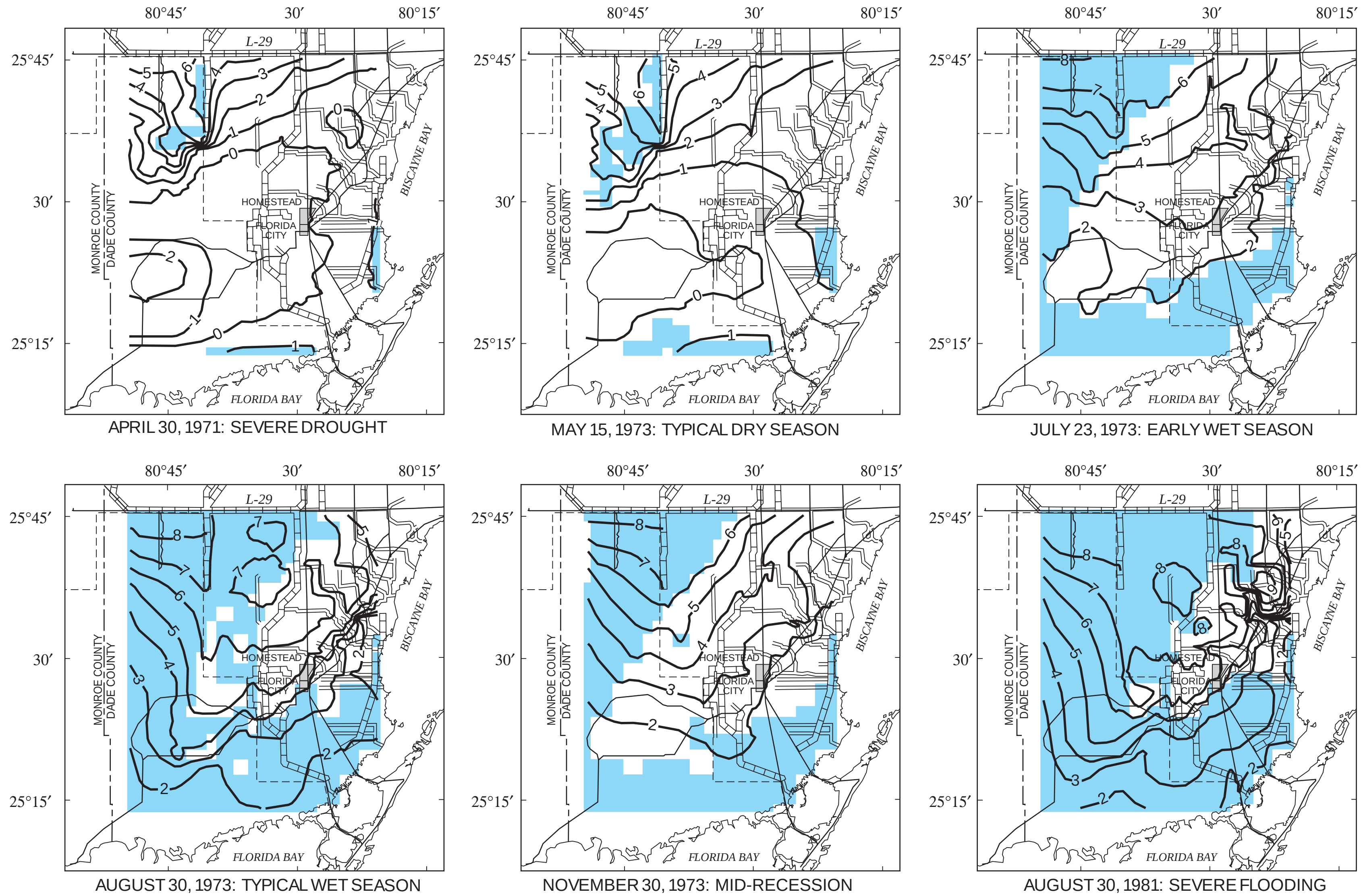
Wet- and Dry-Season Comparisons

The hydrographs of plates 4 to 7 show periods of exceptionally high water in the wet seasons of 1968, 1969, and 1981. The first two summers were marked by higher-than-normal seasonal rainfall. Large quantities of rainfall (over 20 in. at some stations) occurred August 16-18, 1981, during passage through the area of Tropical Storm Dennis. This quantity exceeded that associated with any other tropical disturbance during the overall simulation time period (water years 1945-89) and was followed by comparable quantities of rain in September that were not associated with any organized tropical disturbances. Substantial flooding occurred throughout the area on both occasions in 1981 but was quickly reduced by use of the flood-control system (pls. 4-7).

Computed heads in the study area during the simulated peak of August 30, 1981, shown in figure 41 must be qualified in that the depicted heads are lower than those of the true flood peaks and higher than those after several days of drainage. The area of flooding is depicted as more extensive than in 1963 and generally comparable to that of Hurricane Donna in 1960. Only part of the upper Black Creek Canal basin is shown to be flooded, however, in contrast to September 1960.

The most striking feature of the wet-season water-table contours in the 1968-82 time period is the effect of L-67 Extended in separating stages on its eastern and western sides. The effect is enhanced by the routing of water by the borrow canal on the western side of the levee, which raises simulated water-table altitudes near the lower end of the levee and canal.

The record-low water table for this 15-year period, in May 1971, marked the end of a drought of historic severity that caused saltwater intrusion in coastal well fields already stressed by increasing water demands. Effects of this drought were compared to those of earlier and later droughts by Waller (1985). Heads at some measuring stations are overestimated during this low-water period because of the monthly averaging of input parameters, but the simulation is accurate at most stations. A map of water-table



EXPLANATION

INUNDATED AREA—Shaded area is restricted to area contained within the model area

—2— SIMULATED WATER-TABLE CONTOUR—Shows altitude of water table. Contours are restricted to area contained within the model grid. Contour interval 1 foot. Datum is sea level

BOUNDARY OF EVERGLADES NATIONAL PARK

ROAD

LEVEE AND ADJACENT CANAL

CANAL

0 5 MILES
0 5 KILOMETERS

Figure 41. The water-table altitude and the area of inundation in the study area at selected times, as depicted by the calibrated model for water years 1968-82.

contours in the modeled region at the end of April 1971 is presented in figure 41. The contours depict a water table lower than any previously shown, including the low-water tables of 1945 and 1956 (figs. 32 and 34).

The area in which water-table altitudes are below sea level in May 1971 includes most of the southern part of the modeled region and extends to the upper reaches of Mowry and Princeton Canals. In a section of the lower part of Everglades National Park, computed altitudes are more than 2 ft below sea level. In the upper part of Everglades National Park west of L-67 Extended, computed heads are higher than in 1956. However, on the eastern side of L-67 Extended, heads are lower than in 1956. The stage separation maintained during wet-season inundation by the model representation of the levee persists as a separation of water-table altitudes after most parts of the area become dry because of the low aquifer hydraulic conductivity in this area. The 1971 drought and the typical dry-season low water maps (May 15, 1973) demonstrate the persistence of some surface water west of L-67 Extended even during the most severe drought. During earlier droughts (figs. 32, 34, and 39), the area was depicted as entirely dry.

Most observed ground-water and surface-water head data in the fourth time period are matched relatively well by simulated heads. Exceptions are at G-858 (pl. 5), which might be affected by use of boundary conditions to represent the effect of the Alexander Orr and Southwest Well Fields; G-1183 (pl. 5), which does not show the observed wet-season peaks for unknown reasons; the two L-67 Extended canal stations (pls. 6 and 7); and NP-201 and NP-202 (pl. 7), for which early data might be subject to slight datum errors. In addition, June 1971 rainfall in the southern part of Everglades National Park might have been substantially greater than the Royal Palm Ranger Station value substituted for the Homestead value.

The comparison hydrographs for wetlands stations (plates 4-6; G-620, P-33, P-37, NP-46, NP-62, G-1251, S-18C, and P-36) again demonstrate a previously noted difficulty arising from one aspect of the coupled simulation of surface- and ground-water systems. If the simulated surface-water stage is slightly higher than measured as at P-33 (pl. 4) in April 1971 and April 1974 and at NP-62 (pl. 5) in March 1979 and April 1981, the beginning of the ground-water recession will be substantially delayed or may not even occur, leading to a large divergence between simulated and observed heads when below land surface. An equivalent effect may occur if the average land-surface elevation simulated for the grid cell differs from that in a relatively extensive area near the gaging station or well, perhaps because of uncertainty in determining the land-surface elevation from available data.

A related problem can occur when the areawide rainfall specification seems to be an underestimate at stations in the Everglades, such as at G-620 in the summer months of 1971. The simulated head may not rise appreciably above land surface, even though nearly 1 ft of surface water was actually present. When such an occurrence is followed by dryer weather, as in the winter of 1971-72, the simulated ground-water recession will be steeper than the actual surface-water recession, leading to a substantial divergence of computed heads.

Despite these difficulties, the results from simulating heads in the wetlands were considered to have satisfied the objectives of the study. Figure 41 includes contours of the regional water-table altitude during a typical dry-season (low-water) period (May 15, 1973), the early part of a wet season (July 23, 1973), a typical wet-season (high-water) period (August 30, 1973), and the mid-recession period (November 30, 1973). The maps for May 15 and November 30 show the effect of the L-31W canal in raising the water table at the lower end of the canal. The maps for July 23 and August 30 (fig. 41) show the drainage effect of Princeton Canal and Mowry Canal. The November 30 map shows the effect of C-113 in locally raising the water table and the influence of C-111 above S-18C in increasing water-table altitudes over a larger area.

Simulation of Drainage for Flood Control

The high water tables of 1968 and 1969 seem to be overestimated by the model despite substitution of the lower June 1968 rainfall total from Royal Palm Ranger Station. During the 1968 wet season, the automatic flood-control system had been in operation only a few months when the high water table necessitated the first manual operation of the system to flush floodwaters. At various structures (table 6), automatic controls were switched off, and gates were opened manually and left open for weeks or months. In 1969, manual operation of the system during flood conditions again became necessary. Afterward, the system was not operated manually again until a freak spring rain (more than 10 in.) occurred in April 1979. Manual operation in anticipation of Hurricane David in

September 1979 was discontinued when the storm veered away. Manual operation was deemed necessary twice in the summer of 1980 and twice in the spring of 1981. The next two manual operation periods occurred in August 1981 and in September 1981. Following Tropical Storm Dennis in August, many structures were under water for 1 to 4 days. The earthen plug in C-111 adjacent to S-197 near the coast was removed on each occasion in 1981 by bulldozing the channel past S-197. Manual operation of the canal system occurred twice more in 1982, and the S-197 plug was removed a third time.

Manual openings of the various control structures, when for periods of several weeks or more, were simulated by setting the corresponding specified monthly control elevations equal to zero. Control-structure openings simulated by the model in the 1968-82 time period are shown in figure 42. The illustration shows the specified manual openings of various structures in 1968, 1969, 1980, 1981, and 1982. Other openings were those triggered by the logic of the model representing the automatic operation of various controls, which in many cases were of less than 1 month in duration. Erratic spikes in the hydrographs of the gate openings were caused by oscillations that still occurred despite the coding of logic to stage simulated structure openings over several timesteps, and also indicate that those openings occurred under simulated automatic control.

The high water table of August and September 1981 is simulated well at some locations (pls. 4-7). This is a result of the substitution of the lower monthly rainfall total for September 1981 from the Homestead Field Station and of the specified zero control elevations at most simulated structures. Generally, the measured water-level data show two sharp peaks, each followed by quick drawdowns occurring within days, whereas the model portrays a single peak and a

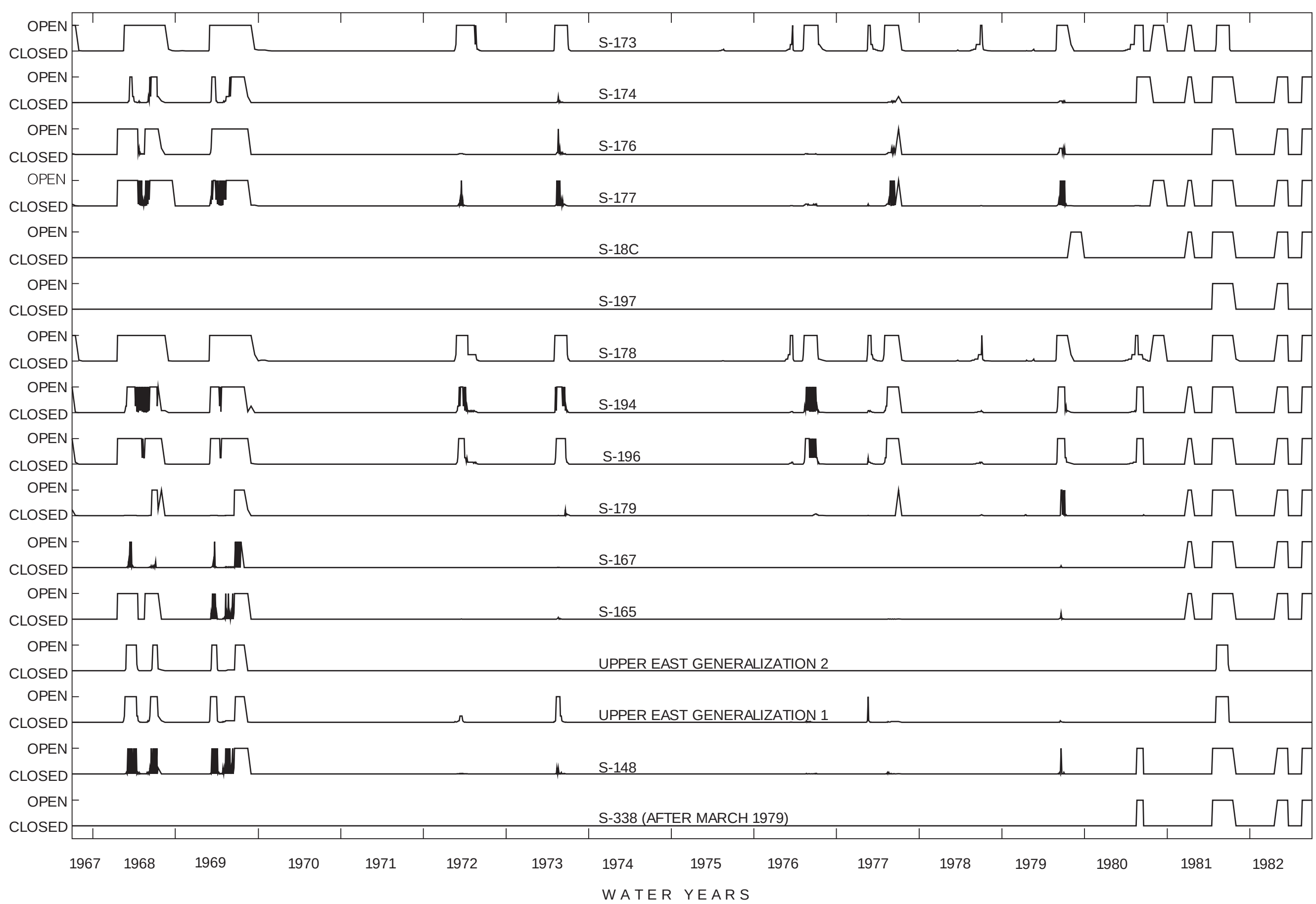


Figure 42. The simulated control-structure openings during water years 1968-82.

2-month recession period. One of the poorer simulations is shown in the hydrograph (pl. 4) for station S-182A near the eastern boundary of the model.

The difficulty in simulating rapid drainage by canals near S-182A was previously cited when evaluating the simulation for time period 3 (water years 1962-67). The same difficulty occurs in simulating drawdowns at many other coastal ridge stations in time period 4 because the system of drainage canals became more extensive after 1967. For example, water-level data measured in well G-614 (pl. 4) during Tropical Storm Dennis in August 1981 show a recession of 4.5 ft in 7 to 10 days, followed by a similar peak and recession in September. The model computes a single peak and requires 2 months to simulate a recession of about 3.5 ft. By contrast, following Hurricane Donna in September 1960, a recession of 2 ft occurred in 2 weeks in G-614 (before Tropical Storm Florence created another high). An additional 2 months were required for the water level to drop 4.5 ft from the peak of Hurricane Donna, and the recession computed by the model was similar. In contrast to the measured water-level behavior in S-182A and G-614 in August and September 1981, measured water levels in well G-1502 (pl. 6), in the rocky glades at a location relatively remote from drainage canals, show a gradual reduction from the 1981 peak that is better simulated by the model.

Failure of the model to represent the rapid drainage of the coastal ridge in flood events of 1981, and also those of 1968 and 1969, indicates an incompatibility between the simulation method designed for this study and the function of the 1968-82 water-management system in reducing exceptionally high water-table altitudes. The principal difficulty probably lies in the monthly averaging of inputs representing rainfall and the manual operation of controls. Increasing the frequency of computational timesteps provided a more realistic simulation in some months but could not completely resolve this difficulty. Most likely, substantially better results could have been obtained by representing weekly or daily rainfall amounts and the corresponding response in the operation of the flood-control system of 1968-82. However, as the simulation objectives of this study were accomplished by the procedures used, additional effort to experiment with input specifications more highly discretized temporally was considered beyond the scope of the study.

Influence of Flood-Control System

To gain better insight into how the representation of the post-1967 water-management system affects the simulation of surface-water and ground-water heads, and to use the model to assess the effect of the system, a sensitivity analysis was made. The entire post-1967 system of canals and control structures was removed, except for L-67 Extended and canal, and the remaining canal-levee system was restored to its state in time period 3 (water years 1962-67). Results are shown as comparison hydrographs at selected stations on plate 7 and as contours of water-table altitude changes computed when the representation of the post-1967 system was deleted from the calibrated model (fig. 43).

In the sensitivity analysis, the principal effect on computed coastal ridge water-table altitudes was to raise and prolong wet-season peaks by a modest amount. Computed water levels at the nodal location of G-1363 are identical to those of the calibrated model in most dry seasons, but exceed those of the calibrated model by as much as 0.6 ft in a typical wet season (August 30, 1973). Because G-1363 is between the upper parts of Princeton and Mowry Canals, the result probably illustrates the regional effect of canal drainage. The well is simulated as being slightly closer to the L-31N borrow canal than it actually is. The water-level differences during the high-water peaks of 1968, 1969, and 1981 are greater, as much as 1.5 ft, illustrating the effect of the drainage that occurred when structures were manually operated during periods of exceptionally intense rainfall.

Computed water levels at G-1502, about 4 mi inland and upgradient of L-31N and its borrow canal, were affected less; following the 1981 peak, the hydrographs are separated by a maximum of about 1 ft. The hydrograph separation at G-1502 tends to follow the water-table peaks, demonstrating the existence of a timelag before wet-season control-structure operation affects the water table at this location. G-864 is located 2.5 mi from control structure S-178 at the head of C-111E, and the comparison shows only slight effects of control-structure operation. Wet-season peaks of 1968, 1969, and 1981 are slightly higher (0.2-0.4 ft) in the sensitivity analysis.

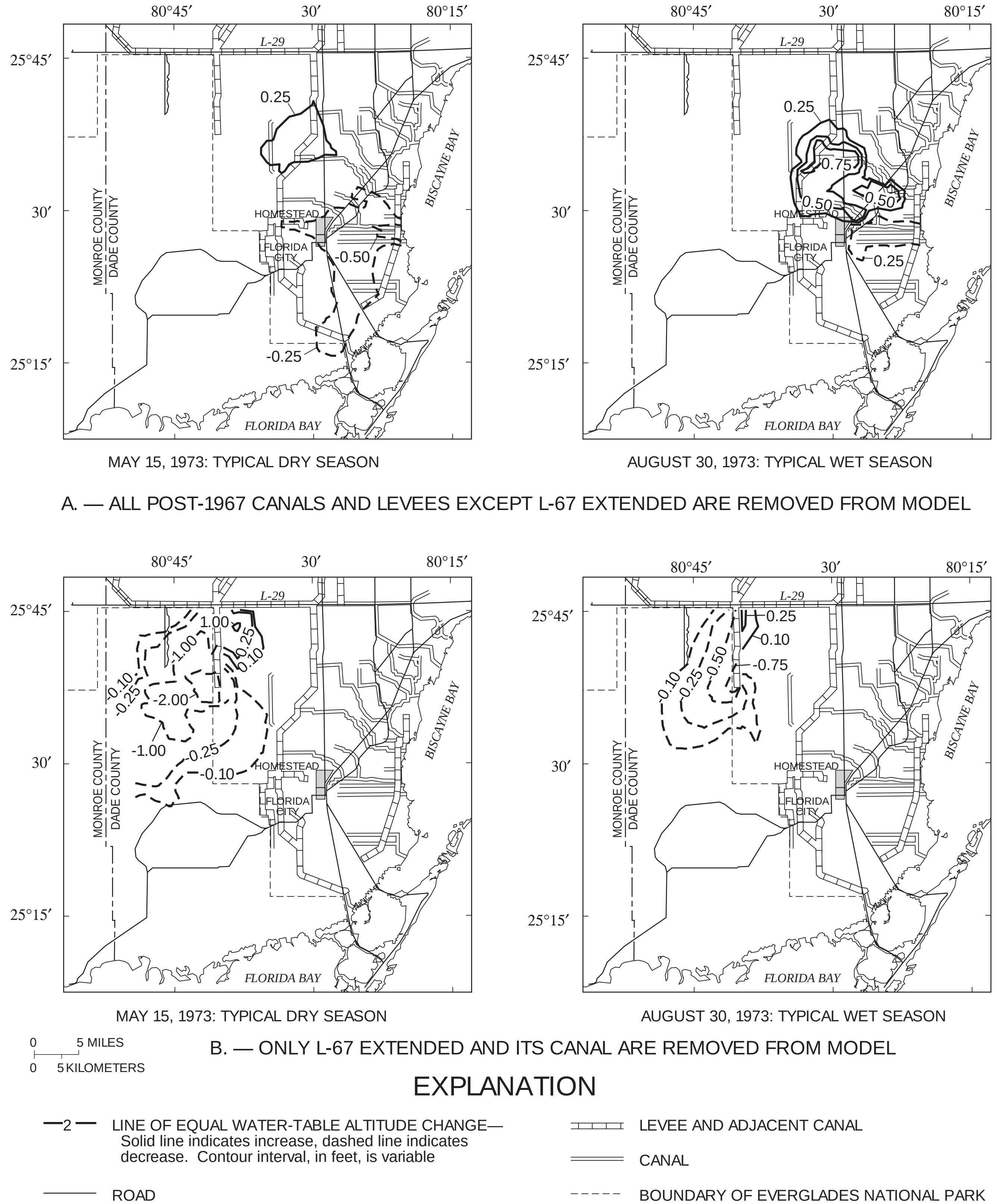


Figure 43. The changes in the altitude of the computed water table at selected times during water years 1968-82 when: (A) most post-1967 canals and controls are removed, and (B) Levee 67 Extended and its canal is removed.

A different effect is evident in the dry seasons, however, when G-864 water levels in the sensitivity analysis are lower than those of the calibrated model. A similar and more pronounced effect is apparent at G-1183 in the eastern coastal lowlands. This demonstrates that the post-1967 canal drainage raises the water table in the vicinity of the downstream reaches.

Contours of water-table altitude changes simulated by the sensitivity analysis (fig. 43) are shown for a day (May 15, 1973) representative of the low water table of a typical dry season, and a day (August 30, 1973) representative of the high water table of a typical wet season. Small areas of higher and lower heads surrounding individual control structures have not been included in the figures so that only regional-scale variations are indicated.

On the dry-season day, differences with the calibrated model are generally not much greater than 0.25 ft. When the canal system is deleted, the water table is higher by this amount in a region in the north-central part of the study area (fig. 43), and lower by a similar amount south and east of Homestead. Because controls are closed on this day in the calibrated model, this probably represents the removal of the effect of local flattening of water-table gradients by sections of the canals between controls.

On the wet-season day, deleting the post-1967 drainage system raises the water table more substantially (0.50 ft and greater) in a region north of Homestead that includes the upper reaches of most of the canals. The water table is slightly lower in the sensitivity analysis in a region east of Homestead that includes the lower reach of Mowry Canal. Many control structures were simulated as open for drainage during August 1973 so the higher water table computed near the upper canal reaches in the sensitivity analysis shows that a significant lowering of the water table was accomplished by use of the drainage system.

Because there is general agreement between measured and computed heads at most observation sites in the 1962-67 and 1968-82 time periods (except for extreme high-water events in the latter time period), and because the sensitivity analysis represents a canal-levee system identical to that of the 1962-67 time period (except for L-67 Extended and canal), the results of this analysis are probably indicative of the actual changes that occurred. The principal purpose of the post-1967 drainage system, the lowering of high water levels during periods of intense rainfall, is indicated by the model to have been achieved.

Influence of Levee 67 Extended

Hydrographs of surface-water gaging stations P-33, west of L-67 Extended, and Northeast Shark River Slough No. 1, east of L-67 Extended (pl. 7), are used to show results of a sensitivity analysis in which the representations of L-67 Extended and its adjacent canal were deleted. Before this could be accomplished, an issue had to be resolved as to whether previously assigned heads along the northern boundary were also reasonable boundary conditions in the sensitivity analysis.

It was assumed that stages in the Tamiami Canal west of the levee would be maintained the same by S-12 structure releases. Nevertheless, the question remained whether the eastward release of surface water by the removal of the impoundment could raise stages in the section of the canal east of L-67 Extended, which was separated from the western section by structure S-12E and from the L-29 canal by an earthen plug until 1978 and by structure S-333 after 1978 (fig. 18). However, the L-67 Extended canal and levee did not exist from 1962 to 1967, the period after construction of L-29, L-67A, and L-67B. During this period, previously cited average monthly stages at Bridge 45 in the eastern section of the canal were actually lower than in time period 4 (water years 1968-82), even though Everglades surface waters could flow north into the canal section. This observation needs to be qualified in that 1962-64 were drought years. However, for purpose of this analysis, canal stages from the eastern canal section in time period 4 were accepted as realistic boundary conditions in the sensitivity analysis.

Results of the analysis indicated that, when surface inundation persisted at P-33, simulated stages were reduced as much as 1 ft. East of the levee at Northeast Shark River Slough No. 1, surface-water stages were raised by a smaller amount, less than 0.4 ft. Head changes computed when the representations of L-67 Extended and its canal were deleted from the calibrated model are shown in figure 43 for a day representative of the high water table of the wet season (August 30, 1973) and a day representative of the low water table of the dry season (May 15, 1973). On the wet-season day, head decreases occur west and south of the now absent levee, and head increases occur east of the upper end of the levee. The computed dry-season changes show this pattern to persist even though surface water has dried. Substantial head decreases of as much as 2 ft are simulated. The effect on surface-water stages and water-table altitudes is negligible east of Grossman Hammock and south of the main park road. The general result of the simulated removal of levee and canal is an eastward spreading and shallowing of surface-water flows released through the S-12 structures.

The result sheds light on earlier analyses of long-term water-level changes in selected wells in successive time periods from water years 1945 to 1989. Statistical comparisons showed average water levels in G-620, bordering Shark River Slough, to be reduced in the 1962-67 time period compared to earlier and later time periods. It is possible that the similarity of 1968-82 water levels in G-620 to those before 1962 is partly the result of the channeling of southward flows by L-67 Extended. Without the presence of the levee and canal, the 1968-82 water table west of L-67 Extended might have more closely resembled the 1962-67 water table in altitude.

Effect of Varying Equivalent Hydraulic Conductivity of Canals

Despite the fact that canal equivalent hydraulic conductivities of 3×10^8 and 2×10^8 ft/d were less than the general estimate of 8.3×10^8 ft/d based on equation 6, calibration results were satisfactory. The calibration was expected to be relatively insensitive to relatively small variations of this parameter, which would cause only small changes in the canal stage that affected the water-table altitude in the surrounding aquifer. To verify this hypothesis, a sensitivity analysis was made in which all equivalent hydraulic conductivity values assigned to canal cell strings were doubled. From equation 6, the effective hydraulic conductivity of a canal could change this much if the hydraulic radius increased by a factor of 2.8 or if the horizontal pressure gradient decreased by a factor of 4. However, temporal changes of such magnitude were unlikely. The principal degree of uncertainty was in the initial estimate of canal equivalent hydraulic conductivity based on an arbitrary selection of a Manning's coefficient value (n) in equation 6.

Results (not shown) indicated computed water-table altitudes in most of the modeled area to be unchanged. Slight water-table altitude changes, generally less than 0.3 ft, were simulated as a shallowing of dry-season low water levels and a lowering of wet-season high water levels in wells on the southern coastal ridge (S-196A, G-595, G-757A, and G-614). At G-596 and G-757A, the simulated high water levels of the 1968, 1969, and 1981 wet seasons were reduced by as much as 0.5 ft. Some of the observed variations were more favorable to a calibration and some were less favorable. Generally, the doubled equivalent hydraulic conductivity values, and perhaps even higher values, would also have produced a satisfactory calibration of the time-period model.

Another result of the sensitivity analysis was the simulated raising of wet-season stages at Everglades surface-water station P-33 by about 0.25 ft. This was attributed to the increased simulated ability of the L-67 Extended canal to channel water south from Tamiami Canal to where the L-67 Extended canal ended near P-33 in Shark River Slough.

The result of the analysis supports a previous conclusion, based on the previously cited sensitivity analysis for the specified value of equivalent hydraulic conductivity of overland flow, that the equivalent hydraulic conductivity values assigned to the canal cell strings are not determined by the calibration with a degree of accuracy that would be considered acceptable for most purposes. Corresponding simulated rates of flow in the canals are largely determined by these equivalent hydraulic conductivity coefficients and are, therefore, not accurately estimated by this study.

Water Budget

The regional water-budget estimates for the fourth water-management time period, water years 1968-82 (table 12), indicate that the ratio of evapotranspiration (-2.124×10^{12} ft^3) to rainfall recharge (2.346×10^{12} ft^3) was 90.6 percent, less than that of the second and third time periods but more than that of the first time period. The net boundary flux, including canal drainage at the coast, amounts to a net outflow of 0.194×10^{12} ft^3, comparable to the difference between rainfall and evapotranspiration (0.222×10^{12} ft^3). The computed water mass balance at the end of the time period was 0.9927. The simulated "total water in place" was $0.365\ 10^{12}$ ft^3 in October 1982 at the end of the time period.

Evaluation of the Simulation for Time Period 5 (Water Years 1983-89)

Measured heads and those computed by the calibrated model for water years 1983-89 (time period 5) are compared on plates 8 and 9. A total of 42 stations provided data from all or most of the time period to verify the simulation. Several problems suggesting inherent limitations in the simulation capability of the model as presently designed were evident during this calibration. One was the previously discussed difficulty of simulating rapid aquifer responses to managed drainage during periods of exceptionally high water. Another problem, noted in discussions of previous time-period calibrations, concerned the discrepancy between summer rainfall totals for the coastal ridge and the inland Everglades during certain years. A third problem concerned the use of flow-rate data from the S-331 pumping station.

As in earlier time periods, the steadily increasing rates of pumping from the Southwest Well Field in the northeastern part of the modeled region and from the Snapper Creek and Alexander Orr Well Fields on the northeastern boundary were not explicitly represented in the model. An implicit representation, the use of lowered local water levels as specified boundary head values, was adequate for the simulation objectives of this study. Water levels in the nearest nonboundary measurement station, G-858 (pl. 8), used for calibration were matched very well during most of the simulation period.

Simulated water levels for well G-1502 in Grossman Hammock and simulated stages for gaging station Northeast Shark River Slough No. 1 (pl. 9) were lower than measured values for most of the time period, even though simulated heads for other nearby stations matched measured data very well. The reason for this is not known.

Rainfall Regions

During the first 4 water years (1983-86) of the time period, monthly rainfall totals from the Homestead Agricultural Experiment Station were used to represent rainfall recharge throughout the modeled area (fig. 29). Because of substantial gaps in the record from this station after 1986, rainfall values from the Homestead Field Station (of the South Florida Water Management District) were used in the remaining water years (1987-89). After other problems were resolved, these data provided a good match of measured stages at gaging stations and water levels in wells in the eastern part of the modeled area, the coastal ridge and the eastern and southeastern glades regions. However, measured heads in the western part of the modeled area, including Shark River Slough, Long Pine Key, and the rocky glades (fig. 3), were substantially underestimated in 1985, 1986, and 1987. Similar poor matches of computed and observed head data in these areas were previously observed in water years 1955-58 and 1971, all years with low summer rainfall as occurred in the years 1985-87.

Somewhat inconclusive evidence was previously cited that summer rainfall tended to concentrate in upper Shark River Slough and the rocky glades in dry years. Therefore, to obtain an accurate regional simulation, the modeled area was divided into two regions in which independent monthly rainfall totals were specified for the summer months of 1985, 1986, and 1987. In these periods, values recorded at NP-203 or in Chekika State Recreation Area were used in the western rainfall region. Values used in each rainfall region during these periods are listed in table 10. Other rainfall substitutions in the 1983-89 time period were few. Three Royal Palm Ranger Station values were substituted for Homestead Agricultural Experiment Station values in the coastal rainfall region. One Chekika State Recreation Area value was substituted for a Homestead Field Station value in September 1988 and was used throughout both rainfall regions.

Two sensitivity analyses were made to show the contrast between the different rainfall specifications used in the two rainfall regions. Comparison hydrographs for well S-196A and surface-water station P-33 on plate 9 show water levels or stages computed using the two rainfall region scenario (the calibrated model), water levels or stages computed by applying Homestead rainfall totals to the entire modeled area, and water levels or stages computed by applying upper Everglades rainfall totals to the entire modeled area.

The difficulty in accepting uniform regional rainfall specifications is evident. When Homestead values were used everywhere, summer stages at P-33 were substantially underestimated in 1985, 1986, and 1987. Effects were similar at other surface-water stations in Shark River Slough and at wells farther south in the higher elevation region of Everglades National Park that includes Long Pine Key. Farther south, stages at P-37 near Taylor Slough were affected to a lesser degree. In contrast, when summer rainfall totals for 1985, 1986, and 1987 were increased uniformly throughout the modeled area to values recorded at the two Everglades stations, computed wet-season water levels at S-196A were too high in 1985, 1986, and in late summer 1987, though they seemed to produce a better match in the first part of the summer of 1987. This effect was replicated at nodal locations of other wells in the coastal ridge.

South Dade Conveyance System Representation

Pumping through S-331 was simulated indirectly by representing a pair of wells in the L-31N canal cell string on either side of the model location of the pumping station and structure S-173. The wells were given equal and opposite rates, equal in absolute value to the estimated pumping or siphoning rate at the structure. Because S-173 was simulated as closed after December 1982, the paired and opposing wells were separated from each other by a barrier in the canal cell string and could interact hydraulically only through aquifer grid cells. Because flow-divide structures S-194, S-196, and S-338 were normally open or partly open, as were structures S-176, S-177, and S-18C in C-111 and S-178 in C-111E (table 4), their representations in the model were assigned zero monthly control elevations throughout the 1983-89 time period. Other structures were assigned nonzero control elevations lower than before 1983. To account for the connection of the L-31N canal to Tamiami Canal as part of the conveyance system, the stage downstream of S-334 in Tamiami Canal was used as a northern boundary condition for the L-31N canal cell string. This helped to maintain realistic stages in the L-31N canal when S-331 pumping occurred. The fundamental scenario represented was that the pumping well in the canal collected water from the canal cross section, from the northern boundary, and from the aquifer. The recharging well in the canal on the opposite side of the barrier supplied equal quantities of water to the canal cross section that either recharged the aquifer or were flushed to the ocean through open control structures.

The equivalent hydraulic conductivity of the L-31N canal was increased by a factor of 10, to 20×10^8 ft/d, to reflect the canal improvements. The coefficient assignments for C-111 remained as before. An examination of simulation results for February 30, 1986, when virtually no pumping or siphoning occurred at S-331 and when S-173 was simulated as being closed, shows that a 0.20-ft drop in stage occurred between S-334 on the northern boundary and the nodal location of S-331. This is the stage loss actually measured in this reach in water years 1984-86 (fig. 24).

Manual openings of the entire system became more frequent after 1983 (table 6). In fact, virtually all simulated openings of the control structures (fig. 44) were specified manual openings. The removal of the S-197 plug during flood conditions in August 1988 was simulated as a manual opening of 1-month duration. The simulated manual openings also represent analogous operations performed by telemetry from the South Florida Water Management District Headquarters in West Palm Beach.

A problem arose when using S-331 flow-rate data. Monthly average wet-season pumping rates were as high as 700 to 800 ft^3/s. When flows of these magnitudes were specified, however, water-table altitudes in the southern coastal ridge were raised to unreasonable levels. On the other hand, reasonable agreement between computed and observed well water levels and canal stages was obtained by limiting average monthly pumping rates to 300 ft^3/s.

According to R. Mireau (South Florida Water Management District, oral commun., 1990), estimated pumping rates could not have been in error to this extent. One possible explanation for the discrepancy between estimated pumping rates and those used for calibration was that canal walls became sufficiently clogged to limit rapid recharge of the aquifer when stages downstream of S-331 were raised substantially during wet-season pumping. If this were to occur, larger volumes of water would flow though C-111 without recharging the aquifer to the extent implied by the model assumption of infinite leakance. However, the clogging hypothesis fails to explain why use of recorded pumping rates higher than 300 ft^3/s draws down canal stages and surrounding aquifer heads upstream of the pumping station more than is shown by measured data.

An alternative explanation was sought by increasing the equivalent hydraulic conductivity of the strings of grid cells representing canal reaches other than L-31N. Because an estimate of canal equivalent hydraulic conductivity based on equation 6 indicated a value of 8.3×10^8 ft/d, increasing the assigned values seemed justifiable. When assigned values were doubled, use of unedited S-331 pumping data did not raise heads in the southern coastal ridge to levels as high as those previously computed. However, drawdowns in the canal and aquifer upstream of the pumping station were still too large when 300 ft^3/s was exceeded, and agreement between simulated and observed heads in the southern coastal ridge was poorer during the entire time period.

A more likely explanation of the poor results using recorded pumping rates is that backward leakage through the aquifer may have occurred near the pumping station. When pumping rates were higher than 300 ft^3/s, the difference in stage upstream and down

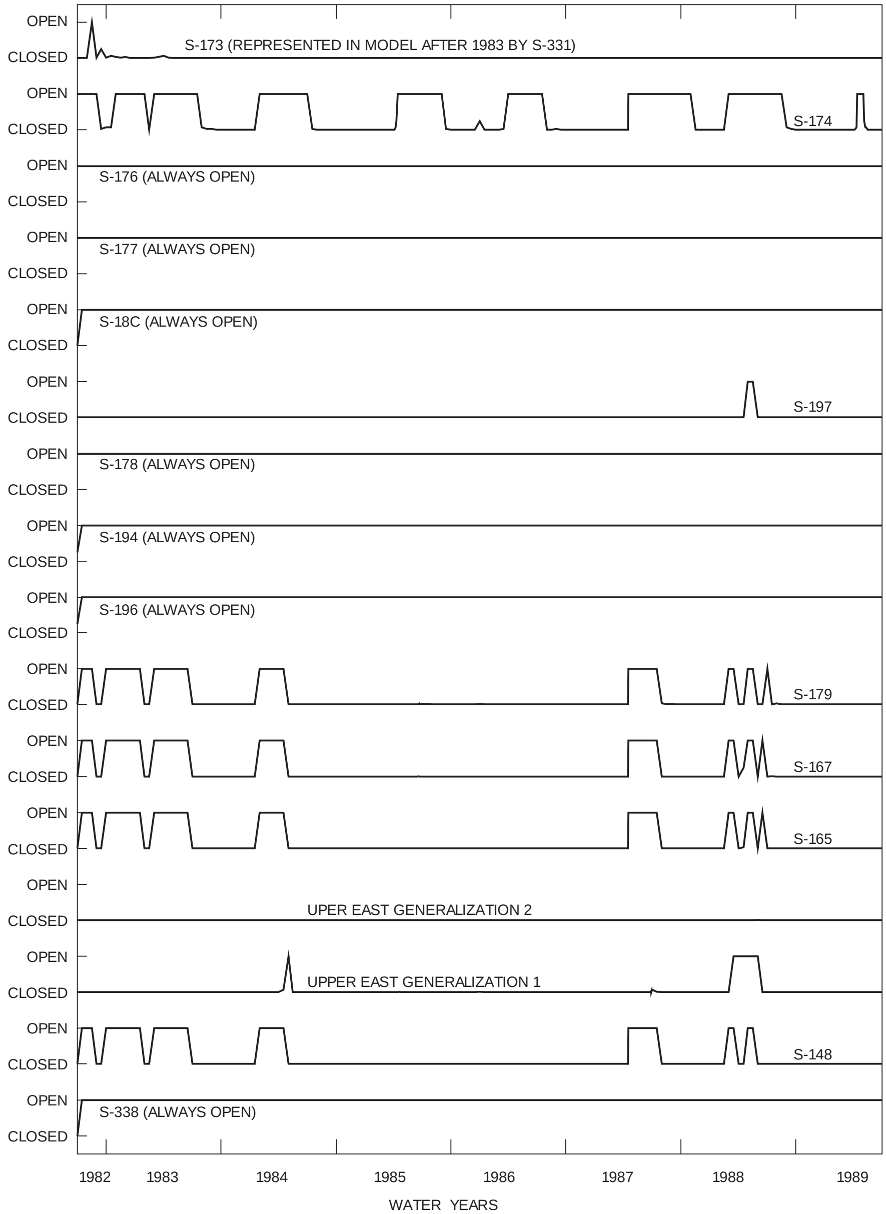


Figure 44. The simulated control-structure openings during water years 1983-89.

stream of the structure, occurring over a distance of a few feet, might have induced backflow and, in time, could also have caused additional solution channel development along the principal flow path. Streamflow data downstream of the pumping station that would confirm that canal flow rates were as high as the recorded pumping rates are lacking. Because S-331 pumping was curtailed in 1991, opportunities to collected such data have become infrequent. Because use of the edited pumping data provided an apparent calibration that replicated heads adequately for the purposes of the simulation (providing boundary condition values for a plume transport simulation), no further effort was expended on efforts to resolve these simulation difficulties.

Wet- and Dry-Season Comparisons

Several weak tropical storms passed near the study area during this 7-year time period, but significant quantities of rainfall were only produced by Tropical Storm Bob in July 1985 (about 4 in.) and Hurricane Floyd in October 1987 (about 2.5 in.). These amounts were small compared to the summer storms of 1988, which generated the highest water-table altitude of the time period. A total of 6 in of rain fell in Homestead in 3 days near the beginning of June and another 8 in. fell in 3 days in mid-August. The control structures were operated manually during both months, and the earthen plug near S-197 was bulldozed in August.

Many hydrographs of water levels measured in wells in the summer of 1988 show two sharp peaks from the storms of early June and mid-August, each followed by a rapid recession of the water table that was aided by manual operation of the flood-control/conveyance system. The model, driven by monthly average inputs of rainfall data, shows a gradual increase to one or two shallow peaks, followed by a gradual recession. As previously indicated, a more exact simulation of the daily variation of measured water levels cannot be achieved using the simulator of this study without a more frequent specification of some input parameters. Computed heads for August 30, 1988, were considered sufficiently representative, however, to display regional contours in figure 45. It should be noted that, for a period of several days, heads in the region of rapid drainage (the southern coastal ridge agricultural area) might have been as much as 3 ft higher than those shown and might have declined rapidly to as much as 2 ft below those shown. A depiction of regional flow velocities in figure 46 shows flows toward all coastal boundaries and a convergence toward major drainage canals.

Between 1983 and 1989, heads in the modeled region never approached record lows that occurred in previous time periods, both because of the operation of the conveyance system and because of the absence of prolonged and severe drought conditions. The lowest water table was that of May 1984. The 1984 water table was not accurately simulated, however, because of the monthly averaging of input data specifications (pls. 8 and 9). Substantial rainfall occurred near the end of the month, and these amounts increased the monthly average rate. Even though May rainfall rates are set equal to one-third the monthly average for the first half of the month, this amount so exceeded the prevailing near-zero rate that heads were overestimated.

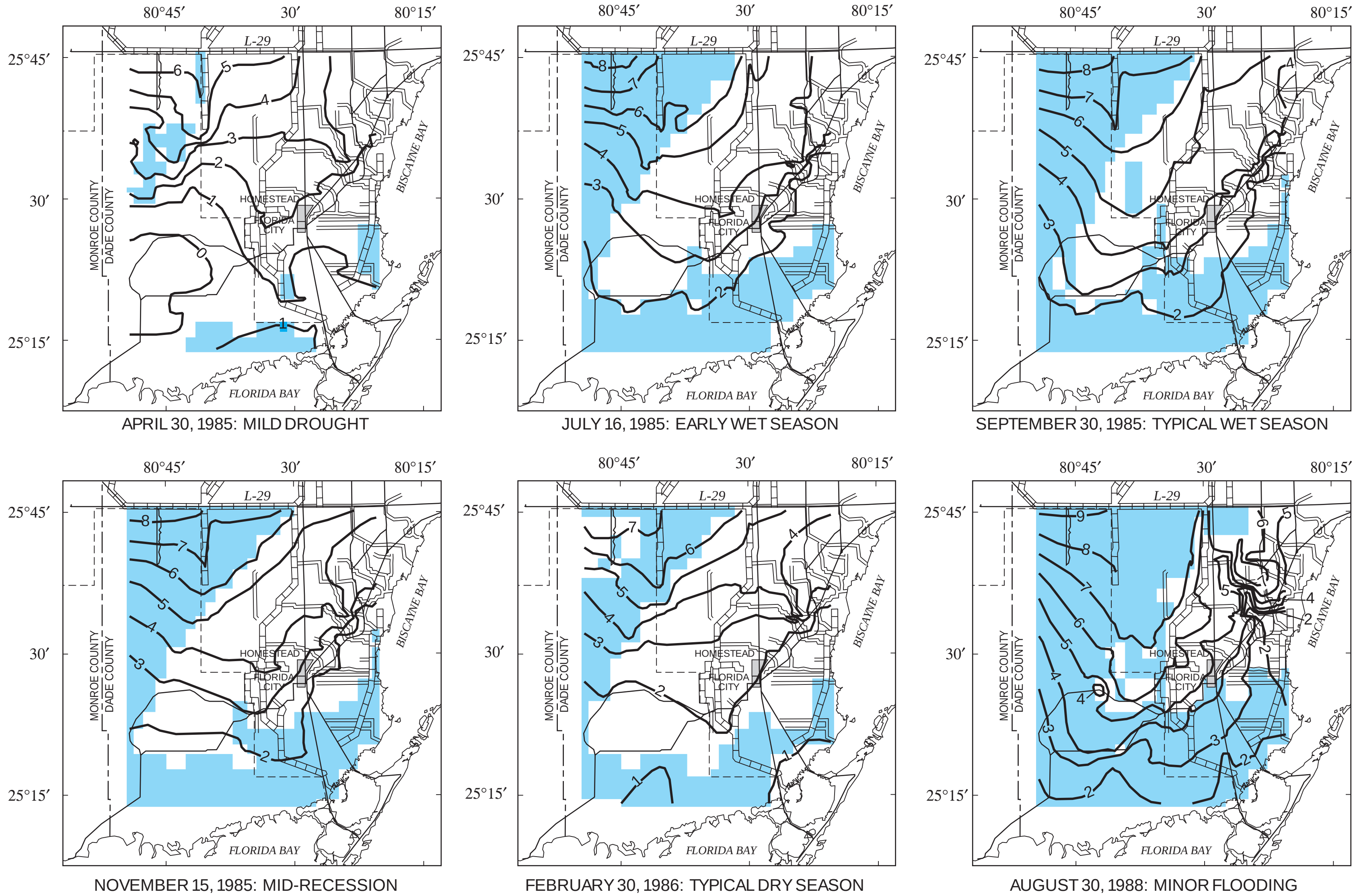
In contrast, the low water table of April 1985 was accurately simulated (pls. 8 and 9), and contours of heads computed for the end of that month are illustrated in figure 45. Heads in a large part of the southwestern Everglades are sufficiently unaffected by the conveyance system to be below sea level. The flow vector display (fig. 46) shows flows in April 1985 converging toward the low-water area. Also shown are eastern and western flows from the upper part of C-111, near which the water table has been raised by operation of the conveyance system.

Contours of heads and flow vector distributions are also presented in figures 45 and 46 for early wet-season conditions (July 16, 1985), typical wet-season high-water conditions (September 30, 1985), mid-recession conditions (November 15, 1985), and typical dry-season low-water conditions (February 30, 1986). The maps show the function of the L-31N canal and C-111 system in collecting aquifer flows in the upper reaches and recharging the aquifer in the lower reaches. The effects of the other drainage canals are noticeable. In particular, Black Creek Canal raises the downstream water table at most of the times illustrated, but is shown to be a major drain during the flooding of August 1988. Strong southeastward drainage now occurs year-round north of Grossman Hammock and west of L-31N and also in the northeastern part of the study area, perhaps influenced by boundary conditions representing the effects of well-field pumping. In the south-central part of the region, strong wet-season flows diminish in the dry season.

Influence of Manual Operation of Flood-Control/Conveyance System

To assess the effect of the practice of manually operating the system of control structures during periods of high water following heavy rainfall, a sensitivity analysis was made. The method of analysis was conceptually simple; the zero control elevations assigned at various controls to represent months of manual openings were replaced with the normal control elevations for those periods. The representation of the manual openings in the calibrated model, required to be a monthly specification, was necessarily somewhat nonprecise. Additionally, simulated openings were represented as full openings, whereas the actual openings are sometimes only partial. Therefore, results of the sensitivity analysis were considered to provide only a general and nonprecise depiction of the effect of the manual openings, though the analysis was precise in illustrating the behavior of the model.

Results are illustrated in hydrographs of computed water levels in well S-196A (pl. 9). This well is simulated in layer 4 of the model directly beneath a string of canal cells in layer 2, but results of the analysis are typical of those in all southern coastal ridge wells.



EXPLANATION

INUNDATED AREA—Shaded area is restricted to area contained within the model area

—2— SIMULATED WATER-TABLE CONTOUR—Shows altitude of water table. Contours are restricted to area contained within the model grid. Contour interval 1 foot. Datum is sea level

BOUNDARY OF EVERGLADES NATIONAL PARK

ROAD

LEVEE AND ADJACENT CANAL

CANAL

0 5 MILES
0 5 KILOMETERS

Figure 45. The water-table altitude and the area of inundation in the study area at selected times, as depicted by the calibrated model for water years 1983-89.

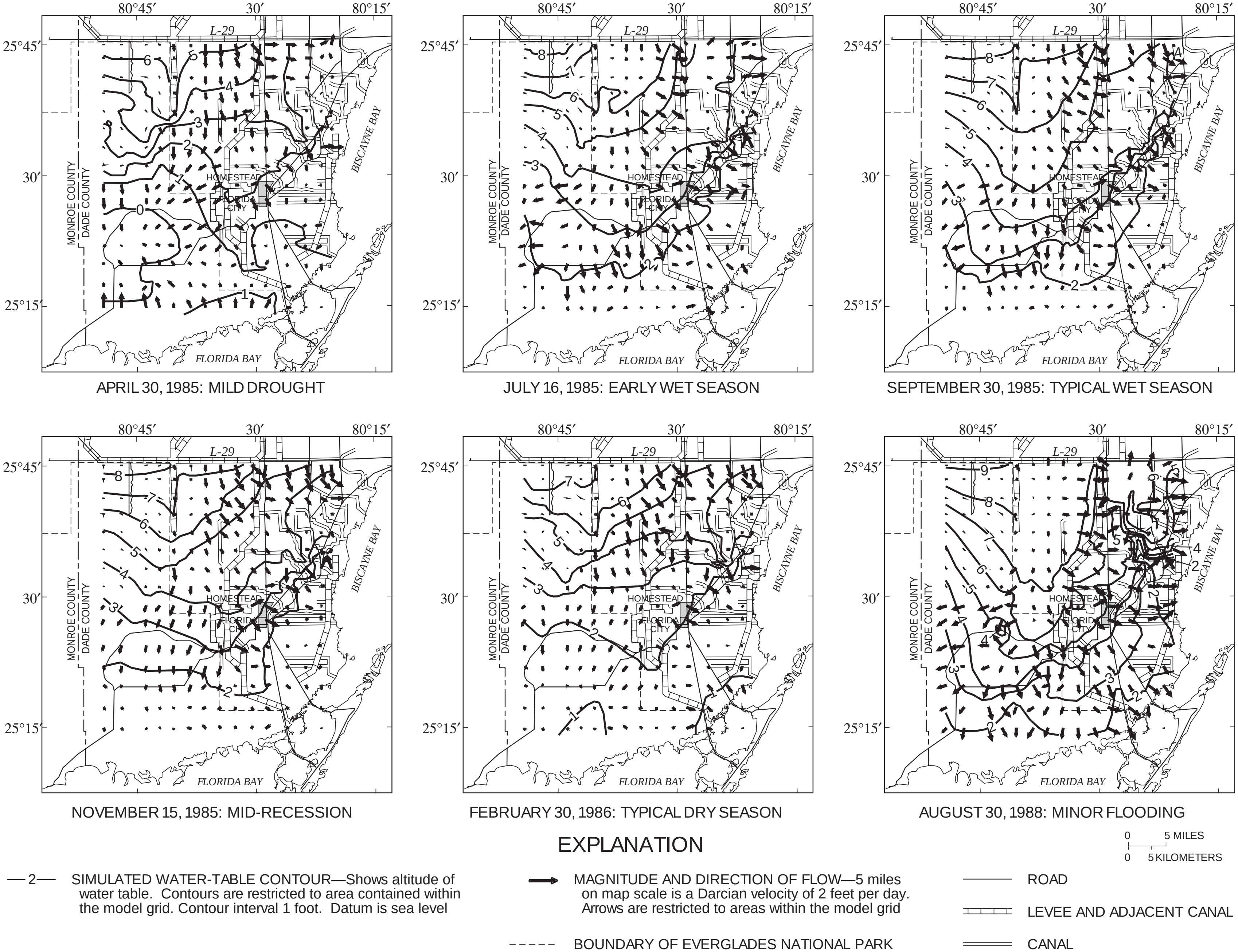


Figure 46. The magnitude and direction of flow in the Biscayne aquifer in the study area at selected times, as depicted by the calibrated model for water years 1983-89.

A reduction in S-196A water levels of as much as 1.5 ft occurs rapidly when manual openings are simulated in the calibrated model, and water levels of the calibrated model quickly rise to the higher levels of the sensitivity analysis when manual openings end and both analyses simulate normal control-structure operation. At a distance from the controlled drainage, as at well G-1502, the effects of manual openings are small and retarded in time. Near the coast, as at well G-1183, results are reversed and a higher water table results from these manual operations.

The analysis depicts the manual openings as being highly effective but as having no long-term effects on the water table when normal automatic operations resume. The comparison hydrograph (pl. 9) indicates that the southern coastal ridge water table might have been considerably higher in the winter of 1982-83 and the summers of 1983, 1984, 1987, and 1988 without the manual openings. When the manual openings were deleted, the simulated annual high water at S-196A in August 1988 exceeded all previously computed yearly peaks at the well since 1969 (although actual heads were higher in 1981).

Influence of South Dade Conveyance System

The model was used to assess the influence of the South Dade Conveyance System on the water table in the modeled area. As previously noted, the system was originally constructed to provide water to southern Dade County during severe dry periods and to increase the capacity of the system for removal of floodwaters, but subsequently has also been used to drain an often-flooded residential area and to prevent flooding of agricultural areas.

The sensitivity analysis was designed by restoring the configuration of the previous (1968-82) flood-control system. The S-331 pumping was deleted as was the representation of S-338. All permanent structure openings (as at S-196, S-194, S-176, S-177, S-178, and S-18C) were deleted, and all control structures were assigned their 1986-82 control elevations. In control structures continuously open in the calibrated model, a rationale was needed for the assignment of times of manual openings. The solution was to represent manual openings as occurring in the same months as they occurred in other structures in the 1983-89 time period (table 6) that were not continuously open in the calibrated model and that had periods of manual operation. The L-31N canal below Tamiami Trail was separated from the Tamiami Canal as in the 1968-82 time period, and its stage was no longer controlled by the measured stage downstream of S-334. Structure S-173 in the L-31N canal was represented as always being closed except for a short time (1-2 weeks) in 1982, blocking southward canal flows to southern Dade County.

Results of the analysis are striking. Comparison hydrographs for S-196A (pl. 9) show that when the conveyance system representation is deleted, water levels are reduced throughout the entire time period as much as 1.5 ft and rarely less than 1 ft. Only when both the calibrated model and sensitivity analysis have manual openings are the two hydrographs nearly the same, as would be expected. At other stations in the southern coastal ridge (F-358, G-613, G-614, G-757A, G-789, G-864, G-1362, G-1363, and G-1486), the reduction of water levels was similar to that at S-196A, although differences are slightly less (0.5-1.0 ft). Away from the southern coastal ridge, the influence of the conveyance system diminishes. Water levels are reduced 0.1 to 0.4 ft at well G-1502 to the northwest and at well NP-206 to the west. At well G-1251 in the southern glades, water levels are not affected when above land surface, but they are reduced by as much as 0.8 ft when below land surface. At well G-3356, water levels are reduced about 0.3 ft when below land surface.

At well G-1183 at Homestead Air Force Base, water levels in the sensitivity analysis were reduced about 0.2 ft. In the northeast, water levels at wells G-855 and G-858 were reduced by about 0.4 ft. In the Everglades north and south of NP-206, the effect on heads is negligible. The stage recorded upstream of S-331 was raised as much as 0.5 ft in the sensitivity analysis. Water levels were slightly higher (0.2-0.3 ft) in some months at wells G-596 and G-1487, suggesting that some of the water pumped through S-331 in the calibration model was removed from storage in the Biscayne aquifer.

The influence of the conveyance system is further illustrated in figure 47, which shows regional water-table changes computed when the conveyance system representation is replaced with the 1968-82 drainage system representation in the calibrated model. The times shown are from a typical wet-season high-water period (September 30, 1985) and a typical dry-season low-water period (February 30, 1986). The wet-season contours show that removal of the conveyance system causes a lowering of the water table throughout a large region that includes the L-31N and

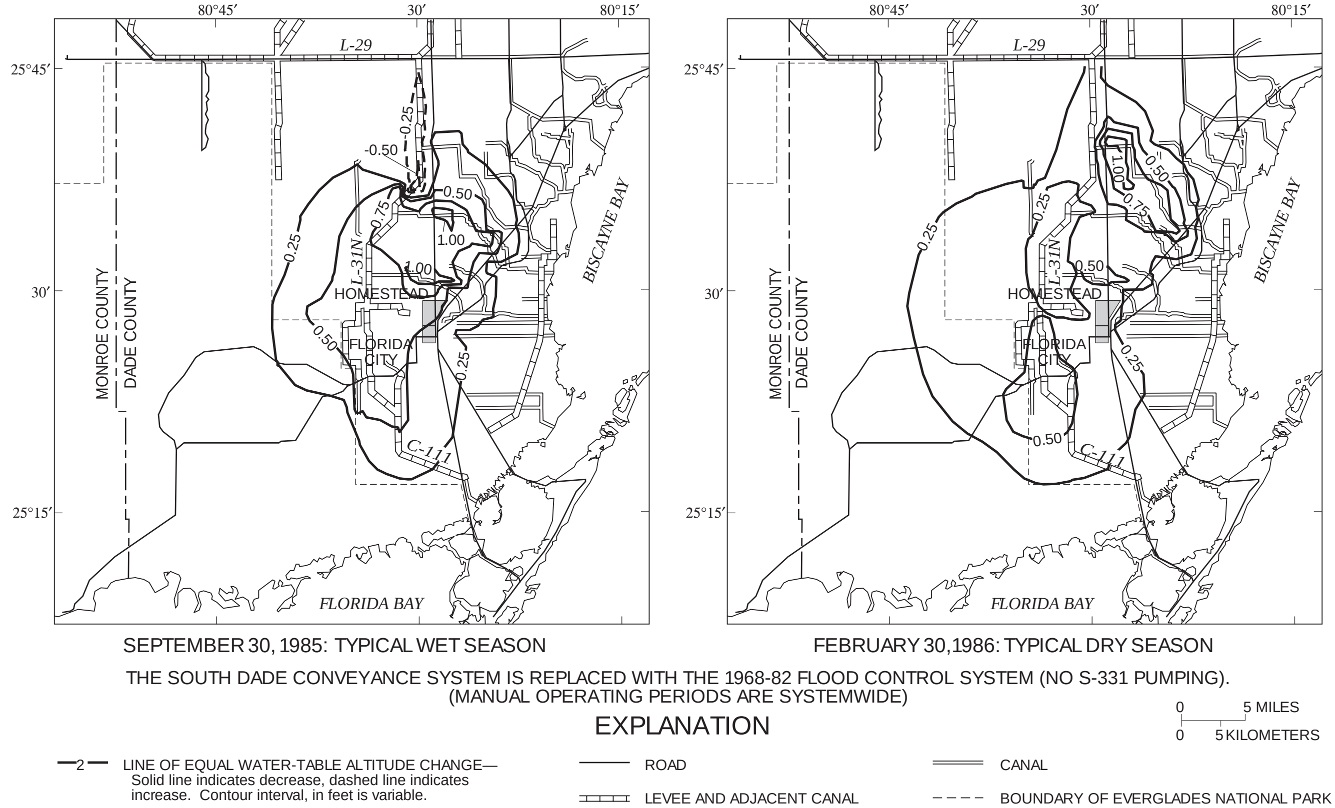


Figure 47. The change in the altitude of the computed water table at selected times when the post-1983 South Dade Conveyance System is replaced in the model with the 1968-82 flood-control system

C-111 basins and the upper reaches of most drainage canals. These are the regions that are recharged by water pumped by the S-331 pumping station to drain areas farther north in the calibrated model. Most of the difference contours pass through the location of S-331 in L-31N and do not enclose the section of L-31 to the north, where heads increase slightly in the sensitivity analysis. Head decreases in the sensitivity analysis are greatest (more than 1 ft) in a region north of Homestead that includes well S-196A.

The dry-season contours show head decreases in the sensitivity analysis to be especially large near upper Black Creek Canal, indicating the effect of deleting the continual releases through S-338, and in the middle C-111 basin, indicating the effect of deleting the continual releases through S-176, S-177, and S-18C. However, the lower L-31N basin shows little change, probably because releases through S-173 are negligible in both calibrated model and sensitivity analysis and because the volumes of releases by dry-season siphoning through S-331 in the calibrated model of the conveyance system are relatively small.

Generally, the analysis indicates that operation of the conveyance system has caused the water table to be raised in parts of south-central Dade County, owing to recharge of the aquifer by the volumes of water pumped through the canal system by S-331. Conditions represented in the sensitivity analysis closely resemble those of the successful calibration in the 1968-82 flood-control period; thus, the sensitivity analysis should also be accurate when the conveyance system is deleted from the 1983-89 time period. This suggests that the calibration achieved in the 1983-89 flood-control/conveyance period was largely based on changes resulting from representation of the conveyance system. The ability of the model to simulate the raising of the water table by the representation of flux into unimpeded canal reaches is vividly illustrated by this exercise. In fact, the simulator may function too well in assuming that only aquifer resistance to flow, and not clogging of canal walls, limits the influx from canal cells.

Effect of Varying Equivalent Hydraulic Conductivity of Canals

In time period 4 (water years 1968-82), doubling the values of equivalent hydraulic conductivity assigned to the canal strings in the calibrated model, 2×10^8 and 3×10^8 ft/d, did not have a significant influence on computed heads. Because structures were open more frequently in time period 5, and the canal system used in more ways for water-management goals, this sensitivity analysis was repeated in the calibrated model for time period 5. S-331 pumping rates were limited to 300 ft^3/s. Equivalent hydraulic conductivity values for L-31N, increased to 20×10^8 ft/d in the calibrated model to represent canal improvement, were not changed further in the sensitivity analysis.

Results of the sensitivity analysis (not illustrated) indicated water levels at wells in the southern coastal ridge (S-196A, G-614, G-757A, G-789, and F-358) to be lowered more significantly (0.3-0.6 ft depending on nearness to canals) than in the time period 4 sensitivity analysis, reflecting the greater use of the canal system and more frequent openings of control structures. Differences from water levels of the time period 5 calibration were continuous rather than occurring only during seasonal high- or low-water periods. In some respects, the lowered simulated water levels at the wells are more favorable to a calibration; in other respects, however, they are less favorable. The revised computed water levels would generally be acceptable as a model calibration. As noted earlier, the simulation based on matching well water levels and surface-water stages does not accurately determine the canal equivalent hydraulic conductivity values or the corresponding flow velocities.

At stations near the lower reaches of the canal system (G-613, G-1251, and C-111 below S-18C), average computed heads of the sensitivity analysis are higher by 0.2 to 0.4 ft. At Everglades surface-water station P-33, computed stages in the sensitivity analysis are less by 0.2 ft, even though they were raised in the time period 4 sensitivity analysis. This is because the L-67 Extended canal is now simulated as being blocked at the northern boundary (the Tamiami Canal) and serves only as a local collector and distributor of overland flows. Water levels in other parts of the modeled region were virtually unchanged in the sensitivity analysis.

Water Budget

The regional water-budget estimates for the fifth water-management time period (table 12) indicate that the ratio of evapotranspiration (-1.007×10^{12} ft^3) to rainfall recharge (1.028×10^{12} ft^3) was 97.9 percent--far higher than estimated in any previous time period. A possible explanation for this anomaly was the raising of the water table in the southern coastal ridge by operation of the conveyance system and importation of large quantities of water from the north. A water table closer to land surface increases the rate of evapotranspiration and the rate of aquifer drainage by ground-water flow to discharge areas. A comparison of average simulated evapotranspiration rates for the five time periods reveals that the 1983-89 rate was higher than in any previous time period except for the 1953-61 time period. Another possible explanation is that rainfall recharge was lower than normal during this time period. The simulated 1983-89 average rainfall was less than for any other time period, although only slightly less than that for time period 3 (water years 1962-67). The combination of the high computed evapotranspiration and the low specified recharge rate apparently explains the high evapotranspiration to rainfall ratio for this time period.

During this time period, the net boundary water flux, which includes the flux through all drainage canals, was a net loss of 0.044×10^{12} ft^3. The total S-331 pumping and siphoning volume was about 0.073×10^{12} ft^3 according to South Florida Water Management District data, but reduction of the pumping rates for calibration reduced this quantity to 0.053×10^{12} ft^3. The computed water mass balance at the end of the time period was 1.0065, indicating that simulated influxes and effluxes were in balance. The simulated “total water in place” at the end of the time period was 0.348×10^{12} ft^3.

Well-Field Pumping Scenarios

It is of interest to determine the response of the calibrated model to the hypothetical development of large well fields at various locations in the modeled region. Recently (1987-92), Dade County conducted a review of plans for development of the proposed West Well Field. Several possible locations were investigated (fig. 2), and the planning study included analysis with digital models that used standard techniques not fully adapted to the hydrologic conditions of Dade County. Application of the simulator modified for use in this study to the well-field problem is useful in illustrating the behavior of the simulator when used for the generic problem of well-field pumping in southern Dade County. Results provide a useful perspective from which to view analyses made by the county.

The proposed well field was first represented at the original proposed location, 1.5 mi west of L-31N and northeast of the Grossman well (fig. 2). This site is no longer under active consideration by the county but provided a useful test case for application of the model. The location corresponded to cell (21,5) in the grid (fig. 29). The original planned pumping capacity for the proposed well field was 140 x 10^6 gal/d. However, environmental considerations may restrict the well-field capacity to as little as 40 x 10^6 gal/d. The analyses of this study, therefore, were made using well-field pumping rates of 40 x 10^6 and 140 x 10^6 gal/d.

Use of the revised SWIP simulator, unlike other codes, permitted a representation of the influence of transient flowing surface water in the Everglades. The revised simulator provided a reasonably satisfactory representation of recharge to the aquifer by the L-31N canal, assumed to be in direct hydraulic connection with the aquifer (or to have an infinitely leaky confining layer). The canal recharge representation was very important because a significant part of the water withdrawn by the well field was assumed to be provided by induced recharge from the L-31N canal.

The well-field pumping was represented in the time period 5 (water years 1983-89) model, because time period 5 most closely resembled water-management conditions that will exist when pumping does occur. An important assumption of the well-field pumping analyses was that stages in the Tamiami Canal used as the northern boundary would remain unchanged by the pumping. In time period 5, the L-31N canal cell string was directly connected to the Tamiami Canal boundary, where heads were prespecified based on stages recorded at S-334 and, therefore, would remain unchanged when well-field pumping occurred. This implied that induced canal recharge to supply the well field was balanced by inflows to the canal cell string as if exactly enough water were supplied to the Tamiami Canal boundary to replace canal water removed by the well field. The amount of water supplied to the canal could be estimated from the computed increase in southward flow at the northern end of the canal cell string. This procedure needs qualification in that surface-water flows computed using Darcy's equation and the equivalent hydraulic conductivity parameter have not been verified in this study. However, comparison of quantities supplied in various test cases provided useful insights into the possible regional influence of proposed well fields.

The effect of well-field pumping at the site west of L-31N is illustrated by comparing hydrographs (pl. 9) for well G-1502 and surface-water station Northeast Slough Shark River Slough No. 2 (locations shown in fig. 21) and by 0.1-, 0.25-, 0.5-, 1.0-, and 2.0-ft drawdown contours (fig. 48) for the two pumping rates. The hydrograph of G-1502 shows that the drawdown of about 0.4 to 0.5 ft remains nearly constant throughout the time period. At the surface-water station, however, a drawdown of 0.2 to 0.3 ft only occurred during recession periods and when the stage was below land surface.

The September 1985 and February 1986 drawdown contours (fig. 48) show that when pumping at 140 x 10^6 gal/d, the 0.25-ft contour extends eastward only 1.6 mi to the L-31N canal. In contrast, the 0.25-ft contour in September 1985 extends about 5 mi north, west, and south. In February 1986, the region enclosed by the 0.25-ft contour is even larger. The relation of the drawdown contours to the location of L-31N shows the importance of the L-31N canal as a source of recharge. A comparison of flow rates in the northernmost canal grid cell with those of the calibration model shows an additional 88 x 10^6 gal/d flowing south in September 1985 and an additional 96 x 10^6 gal/d flowing south in February 1986. These amounts are more than 50 percent of the simulated withdrawals from the well field.

In a second sensitivity analysis, the West Well Field was moved to a site about 1.5 mi east of the L-31N canal at the same latitude as before, a location that corresponds to grid cell (25,5). As in the previous analysis, the 0.25-ft drawdown contours (fig. 49) extended westward only to the canal. The 0.25-ft contours for September 1985 and February 1986 are similar and extend about 2.5 mi northeast, 3.5 mi southwest, and about 8.5 mi southeast. The elongated southeastward shapes of the contours probably are influenced by adjacent canals, and the smaller size of the enclosed region indicates that even more water than before may be imported by canals to the well field in its new location. In this location, the well field is bounded on its northern and eastern side by sections of the simulated Black Creek Canal, represented as being continually open through structure S-338 to the L-31N canal, and receives substantially more recharge from the canal system than in the previous case. This is verified by comparing computed flows in the northernmost canal cell with those of the calibrated model. In September 1986, an additional 113 x 10^6 gal/d flows south when the well field is located east of the levee. In February 1986, an additional 116 x 10^6 gal/d flows south.

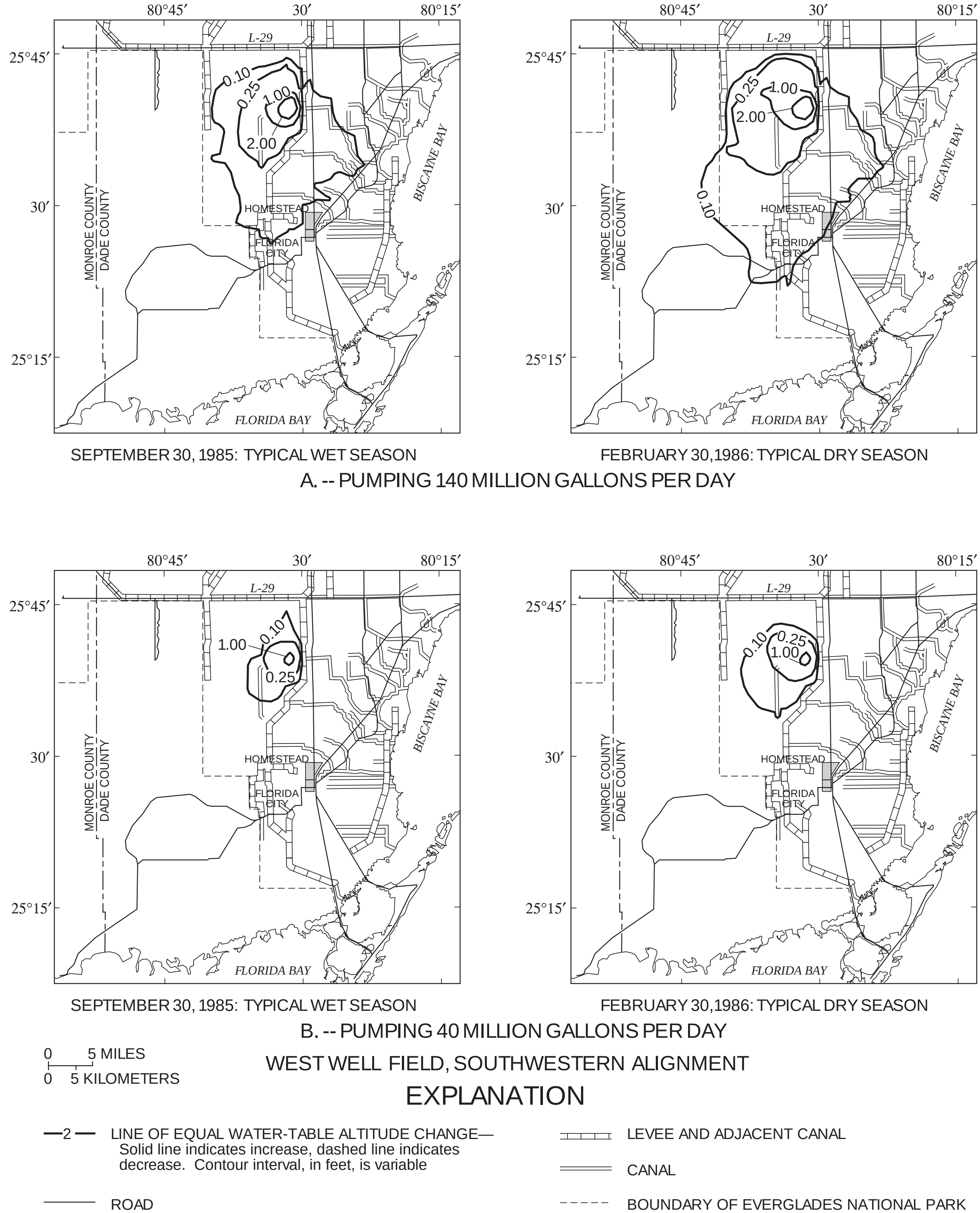
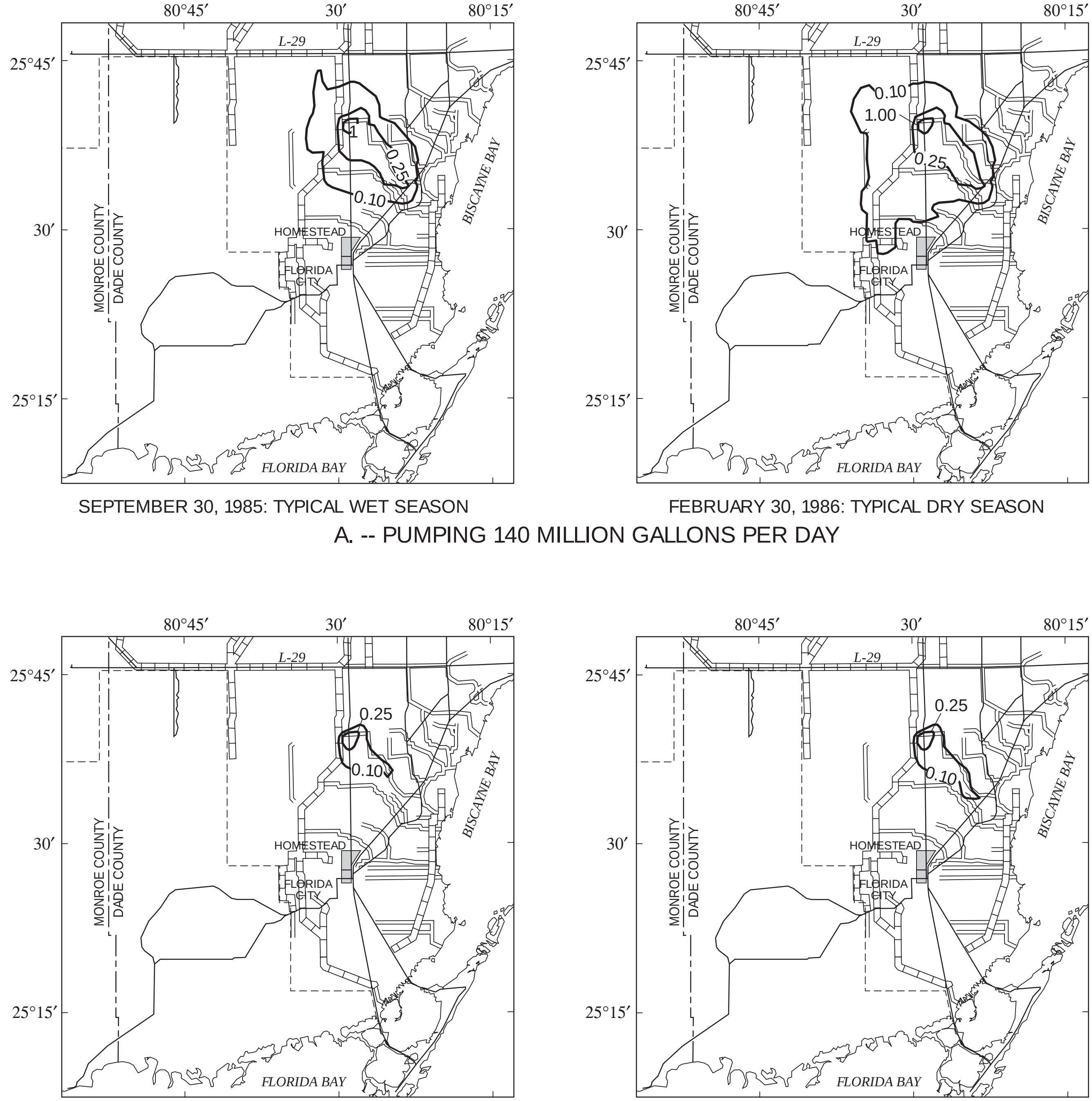


Figure 48. Drawdowns in the Biscayne aquifer when well-field pumping is specified in one of the alignments west of Levee 31N proposed for the West Well Field.



SEPTEMBER 30, 1985: TYPICAL WET SEASON

FEBRUARY 30, 1986: TYPICAL DRY SEASON

A. -- PUMPING 140 MILLION GALLONS PER DAY

SEPTEMBER 30, 1985: TYPICAL WET SEASON

FEBRUARY 30, 1986: TYPICAL DRY SEASON

B. -- PUMPING 40 MILLION GALLONS PER DAY

0 5 MILES
0 5 KILOMETERS

WEST WELL FIELD, SOUTHEASTERN ALIGNMENT

EXPLANATION

—2— LINE OF EQUAL WATER-TABLE ALTITUDE CHANGE—Solid line indicates increase, dashed line indicates decrease. Contour interval, in feet, is variable

ROAD

LEVEE AND ADJACENT CANAL

CANAL

BOUNDARY OF EVERGLADES NATIONAL PARK

Figure 49. Drawdowns in the Biscayne aquifer when well-field pumping is specified in one of the alignments east of Levee 31N proposed for the West Well Field.

The model was also used to simulate the hydraulic influence of a well field located farther south in Dade County. For many years, the county had tentative plans for a South Well Field to be generally located north of Homestead in the southern coastal ridge area, although at the present time (1995), it seems that environmental considerations may cause these plans to be abandoned. For purposes of this analysis, an arbitrary well-field location was selected about 0.5 mi west of L-31N and 6 mi south of Grossman Hammock, a position corresponding to grid cell (15,16) in figure 29. In the early 1990's, some land was under cultivation west of the levee to the north and south of the hypothetical site, but the selected location generally was hydraulically upgradient of agricultural activity.

Pumping at rates of 140 x 10^6 and 40 x 10^6 gal/d was simulated with results (fig. 50) that differed qualitatively from those of previous well-field pumping scenarios. In September 1985, the 1-ft drawdown contour corresponding to the 140 x 10^6 gal/d rate does not extend eastward past the L-31N canal, and in February 1986 the 2-ft drawdown contour is bounded by the canal. In both periods, however, the 0.25-ft contour extends eastward far beyond the canal and encloses a region much larger than that in the West Well Field scenario. What is indicated is that the additional water supplied to the well field by the canal is less than could be supplied to the West Well Field.

In fact, no additional water can be supplied to the canal reach below S-331 in the simulation because the quantity supplied to that reach is specified by the assigned S-331 pumping rates (0-300 ft^3/s), which remained unchanged in the sensitivity analysis. Additional water is supplied by the canal to the drawn-down aquifer north of S-331, as indicated by the velocities in the northernmost canal grid cell, which show an increase of 27 x 10^6 gal/d in September 1985 and 34 x 10^6 gal/d in February 1986 when the hypothetical South Well Field is pumped at 140 x 10^6 gal/d. The rest of the well-field water either comes from the specified volume pumped through S-331 into the lower part of the L-31N canal or from the Biscayne aquifer, and this explains the substantially larger areas enclosed by the drawdown contours. If the region were pristine and no water-management canals existed, all water pumped would come from the aquifer, and drawdown contours would probably encompass even larger areas. In reality, S-331 pumping rates would probably be increased to supply a well field at this location, particularly those lower rates that could be increased without causing substantial ground-water leakage around the pumping station. Drawdown contours would enclosed smaller areas than those shown.

The results of the well-field sensitivity analyses all assume that the aquifer can be recharged by large quantities of water supplied by the L-31N canal. Although this is undoubtedly true, previous evidence indicated that the infinite leakance assumption (no confining layer and no clogging of canal walls) might not be entirely correct when amounts of recharge are large. Results of the well-field analyses must be qualified on this basis, and the influence of possible canal wall clogging on cited results would be to reduce canal recharge and require the well field to remove more water from aquifer storage. The drawdown contours would encompass larger areas than those depicted, and drawdowns at specific location would be greater.

Potential Application for Simulator and for Calibrated Time-Period Simulations

A question inevitably asked by local water managers, given the availability of a simulator with new capabilities or the development of a new model calibration of a significant hydrologic system, is what further uses can be made with the products of these endeavors. In addition, the techniques might have transfer value beyond the local area and potential for further adaptation to other uses. The purpose of this section is to clarify the potential for further work based on results of this study. To accomplish this purpose, it is necessary to categorize the discussion into two separate topics: (1) the potential for further application of the revised model code developed for this study or the potential for further refinement of the techniques developed as part of the code, and (2) the potential for specific further use of the five calibrated water-management time-period simulations.

The model code, now including techniques for representing overland sheetflow, rewetting, canal flows, and control-structure operation, should be generally applicable to the computation of aquifer flows and heads and surface-water stages (but not necessarily flows) in regions similar to the study area, as in other parts of peninsular Florida, and to subregions of the study area. The primary technical limitations that have been noted relate to: (1) the possible need for more frequent (perhaps daily) specification of input parameters; and (2) the lack of any simulated confinement between canals and the aquifer, which might cause the simulator to overestimate rates of aquifer recharge or to overestimate rates of flow from the aquifer to the canal.

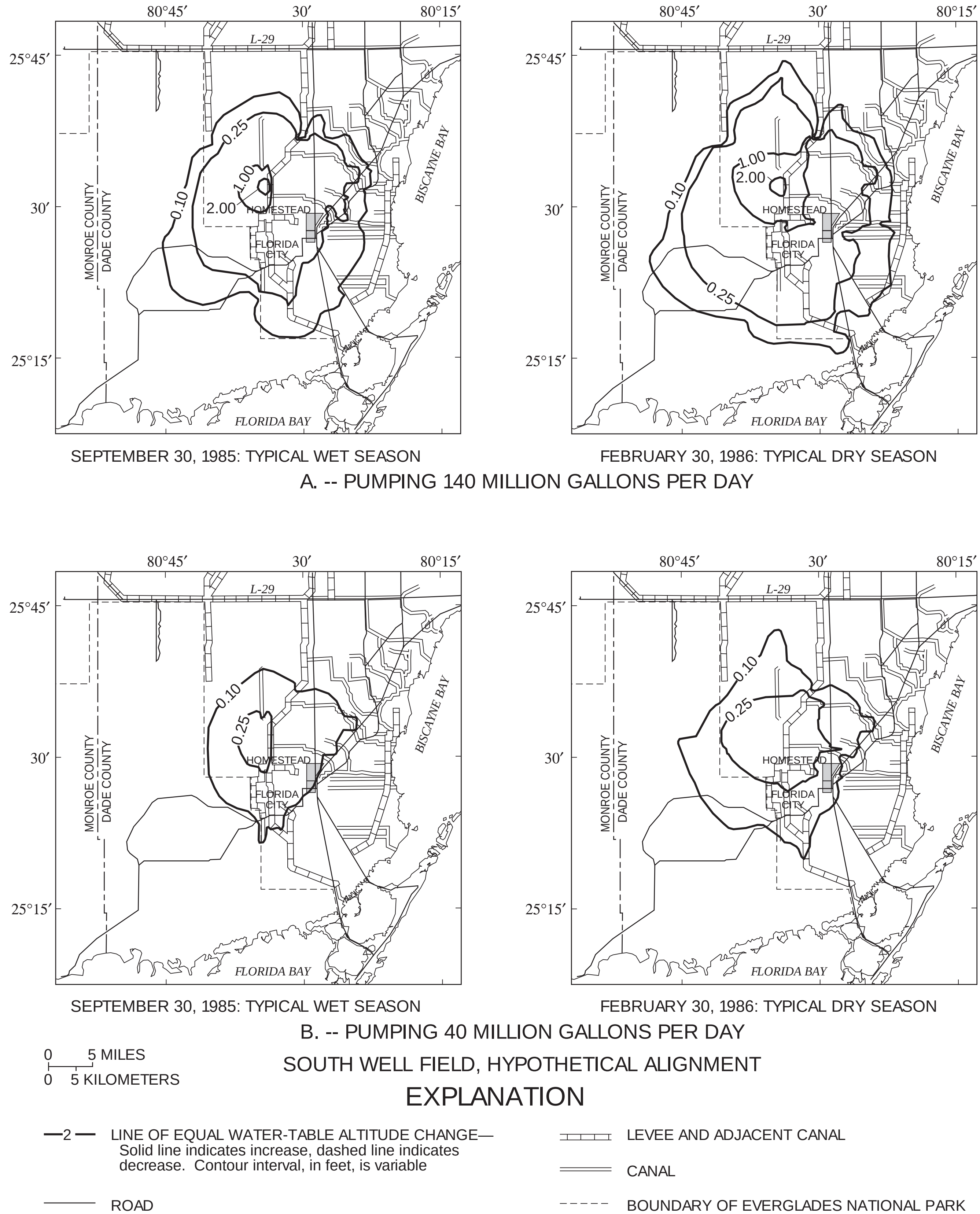


Figure 50. Drawdowns in the Biscayne aquifer when well-field pumping is specified in a hypothetical alignment assumed for a future South Well Field.

The need for more frequent input specifications would depend on the nature of the application problem. Such a capability could be useful where efficient water-management drainage systems quickly lower the water table. Where a data base is available, the input of daily specifications for rainfall and structure control elevations would merely require the recoding of input routines and formatting of the data. Computational time increments could not exceed 1 day, or extra coding would have to be added to accumulate the daily input into sums for larger time increments. Daily calculations might entail substantial computation times. The five time-period simulations were done with 3-day time increments comprising about 50 percent of the simulation time periods and 15-day time increments comprising the other 50 percent. On the fastest local computing equipment available to the Miami USGS office, the Data General file server, the 15-year time period 4 simulation required about 6 hours of computation time (turnaround or clock time). The 7-year time period 5 simulation required about 2 hours. Using 1-day computational timesteps might require about 10 times the previous computation times. Additional testing of the models of the present study with daily inputs might be a worthwhile endeavor in terms of improving techniques, but the data manipulation required might be substantial.

Preliminary responses to the results of this study from local water managers in southern Florida suggest an interest in the model computation of surface-water flow rates, particularly within or on the boundaries of wetlands. As noted, there are mathematical limitations on the use of the algorithm for Darcian flow to compute surface-water flow rates, although this procedure might be justified in wetlands where flow encounters high resistance and is highly channeled. The present method of representing surface-water bodies is an artifice that enables the model to represent the effect of the wetlands and canals in acting as local water-routing mechanisms while their stages vary with, and closely resemble, the water-table surface. A constraint of the present method is that the coefficients representing the resistance to surface-water flow (the equivalent hydraulic conductivities) are not accurately determined by the calibration on measured water-table altitudes and surface-water stages. This means that the surface-water flow rates are also not determined accurately.

These problems could be partly addressed by the acquisition of surface-water flow-rate data. Such data could be used to refine estimates of the values of equivalent hydraulic conductivities and to test the range of validity of the Darcian algorithm. In fact, such data would be needed for the calibration of any available surface-water flow model of the study area. Another effort that might prove to be productive would be to improve the present surface-water flow computation method. It might be possible to develop an algorithm for estimating surface-water flow rates that do not have a linear dependence of the hydraulic slope, possibly using an iterative approach linking independent surface-water and ground-water head and flow computations. A comparison of Darcian and Mannings flow-rate computations for hypothetical surface-water channels might help to determine empirical limits within which their results would be equivalent within an acceptable tolerance.

The possible need to specify some confinement between canal and aquifer might apply only to a narrow range of conditions, such as those following abrupt large changes in the relation between canal stages and aquifer heads. The nature of such conditions would need to be determined through analysis of field data. Possibly, a simple algorithm could be developed to represent canal wall clogging if it does occur under certain conditions.

The possibility that ground-water leakage around structures could explain the necessity of limiting simulated S-331 pumping rates has been noted. Specific efforts to investigate this might include fine-detail subregional models around control structures or the collection of additional field data.

The techniques for representing the hydraulic relation between surface water and ground water in this model were developed for use in southern Dade County, where canals are cut directly into permeable limestone of the Biscayne aquifer and where overland sheetflow is separated from limestone rock by a few inches or feet of peat or marl that does not effectively hydraulically separate surface-water stages and water-table altitudes. However, the new techniques are not necessarily restricted to these hydrologic conditions and might be applicable in other types of environments where surface water and ground water are not fully separated hydraulically by effective confining layers or where a hydraulic separation can be explicitly represented. For instance, where canals are cut into sands or sandstones of lower permeability than the underlying limestones of the Biscayne aquifer, as in parts of Broward and eastern Palm Beach Counties, a model design similar to the one of this study might be appropriate, given the appropriate layer representation and hydraulic conductivity coefficient assignments. It would be necessary to establish that there is no significant confining layer or clogging process separating canals from the materials in which they are completed. This might require field study of the relation between canal and aquifer heads in the specific hydrologic environment.

In addition, where peat or marls separating aquifers from flowing surface water are thick and confining, an explicit representation of the confining layer, as done in this study might still be appropriate, but surface-water stages might not be useful as boundary conditions. However, if significant confining effects separating surface water and ground water under normal hydrologic conditions are the predominant feature of the hydrologic environment, conventional modeling techniques and previously available simulators might be more appropriate for study objectives.

Applications of these techniques to subregions of the modeled region of this study can be facilitated by using heads computed in a simulation of the larger region as boundary conditions for the subregional model, after verifying that computed heads in the appropriate region of the larger model are sufficiently accurate. If the subregional application were to include recharge by a canal, then the relation between canal stage and ground-water head should be investigated to determine if the model assumption of infinite leakance is appropriate under the conditions simulated.

Coastal discharges in the model application of this study were directly specified based on measured data by the artifice of using wells. The simulations might have had greater flexibility of application to a wider range of problems if the coastal discharges had been simulated as control gates governed by stage criteria, like inland control structures. This could be a useful refinement of the techniques used in the present model.

The potential for further use of the five water-management time-period simulations of this study is more limited than the potential for further application of the revised model code and the new techniques incorporated into the code. To construct a regional model that was sufficiently economical in its requirements for computer resources, aspects of the water-management system (canal locations, aquifer properties, well-field pumping, and atmospheric influences) were generalized, especially where precise representation was unnecessary for the principal purpose of the simulation. Furthermore, the simulations were generalized temporally by the monthly averaging of time-varying parameters, including rainfall recharge and control-structure operation. Thus, detailed interpretations of simulation results at various locations in the modeled region and at particular times are not warranted without careful qualification, and the regional model is not well suited for additional use to provide detailed, small-scale, local interpretations. The latter objective might be better achieved by the development of subregional models, using the regional model solely to provide boundary conditions.

Additional applications of the calibrated simulations that are restricted to addressing questions that can be posed and resolved in the generalized regional framework of the simulations, like the sensitivity analyses reported herein, might be achievable with modified versions of the water-management time-period models. One criterion for such application is that conditions represented should not render invalid specific inputs or boundary conditions of the present models, such as rainfall recharge rates, coastal discharge rates, and control-structure operation.

SUMMARY

Increasing demand and the potential for contamination have limited options available for supplying water to the public of southern Dade County, Fla., where the Biscayne aquifer, a designated sole source of water supply, in many places extends to land surface and can be easily contaminated by surface spills. From 1986 to 1992, Dade County cooperated with Federal, State, and other local agencies to select a site for a new well field, designated the West Well Field. One proposed site for the well field was in the East Everglades wetlands west of L-31N. However, the presence of a plume of brackish water, originating from a flowing well in Chekika State Recreation Area, was considered to be a potential source of contamination of the proposed well field. After the well was plugged in 1985, the U.S. Geological Survey, in cooperation with the Metropolitan Dade County Department of Environmental Resources Management, began a study to assess the current extent and probable future movement and dispersal of the plume and the possibility that the plume might interact with the well field. In order to use digital modeling techniques for a simulation of the plume, a realistic representation of the flow regime in the area surrounding the plume was needed to define the direction of its movement. Thus, a major and early focus of the study became the development and application of a simulator that adequately represented the unique hydraulic regime of the area.

In southern Dade County, the Biscayne aquifer, the upper part of the surficial aquifer system, extends to as much as 100 ft below land surface and consists principally of the Miami Limestone and the Fort Thompson Formation. In the western, southern, and eastern parts of Dade County, which are seasonally inundated wetlands, the aquifer is overlain by a layer of peat or calcitic mud that is a few feet thick and of low permeability. Where the Biscayne aquifer has significant thickness in Dade County, it exhibits considerable solution porosity, and the average hydraulic conductivity is estimated to be tens of thousands feet per day.

Natural surface-water flows have occurred in southern Dade County as: (1) overland sheetflow in areas of low land elevation (the Everglades and southeastern glades), (2) episodic streams (transverse glades) that formerly drained the coastal ridge during high-water stages, and (3) perennial streams that drain the southern wetlands. Broad, shallow, marine coastal embayments (Biscayne and Florida Bays) act as hydraulic controls on surface-water and ground-water flows inland and receive surface flows from the southern glades. Average tidal stages increase southward and westward along the coast and vary annually between a peak in October and a low occurring between January and April.

Before development, natural overland flow in the northwestern part of the study area (Shark River Slough) was southward past the Tamiami Trail, curving southwestward toward coastal estuaries. Though controlled at the Tamiami Trail since 1962, the flow regime remains similar to its predevelopment state. The region of widespread inundation and surface flow in the Everglades and southern glades diminishes in area in the dry season and virtually ceases to exist during droughts. Overland flow is rarely deeper than 2 ft and encounters substantial resistance from dense vegetation and a rough and irregular surface. No known attempts to measure surface flow in the Everglades by direct means have been successful.

Data collected daily from gaging stations and wells were compared in the glades regions where overland and ground-water flows were separated by a layer of peat or marl of low permeability. When both surface-water and ground-water heads were above land surface, the comparisons revealed close correlation between short-term responses to atmospheric stresses, indicating that, on a daily time scale, surface water and ground water were not separated hydraulically by the peaty or marly layer. Recharge of the aquifer and the maintenance of water-table altitudes equal to surface-water stages are important to the ground-water budget and need representation in a regional flow model.

Rainfall is the principal source of recharge to the Biscayne aquifer, and evapotranspiration is the principal process of withdrawal. Rainfall in southern Dade County averaged 62.25 in/yr between 1940 and 1992 and ranged from a low of 37.0 in. in 1971 to a high of 94.1 in. in 1947. Between 75 and 80 percent of annual rainfall usually occurred between May and October. December had the lowest average rainfall, and June and September had the highest average rainfall. Studies of evapotranspiration rates under controlled conditions have revealed a close correlation with solar radiation and a corresponding strong seasonal variation.

Early water-management efforts in southern peninsular Florida focused on drainage of Lake Okeechobee to protect farmland to its south. Major drainage canals extended from the lake to the east and west coasts. The Tamiami Canal was excavated between 1916 and 1928 to provide roadfill material. Severe floods in 1946, 1947, and 1948 brought about the creation of the Central and Southern Florida Flood Control District in 1949. One of the first projects of the district, in cooperation with the U.S. Army Corps of Engineers, was the construction of a north-south levee, including L-30 and L-31N, to prevent flooding of the east coast by waters from the Everglades. In 1962, completion of L-29 along Tamiami Trail nearly completed the closure of the central Everglades and made possible their use as water-conservation areas. Controlled flows through the four S-12 structures along a western segment of the levee were the means of releasing flowing surface waters into Everglades National Park to the south of the levee.

The continuing needs for further flood control in southern Dade County were addressed by the construction of an extensive system of canals and levees, largely completed by late 1967. A system of flow-divide, stage-divide, and salinity-control structures was built to control releases, separate stages, and prevent saltwater intrusion. In the late 1970's improvements were made to sections of the Tamiami Canal and the L-30 and L-31N borrow canals, and additional structures were built to control releases. Pump station S-331 was built to force southward movement of flows in the L-31N canal to supply water to Taylor Slough and regional wetlands.

The interrelation between canal stages and local water-table altitudes was investigated by comparing data from several existing pairs of adjacent canal stage and well recorders. The canal stages and well water levels were similar at all times and their variations were closely correlated. Periods when differences between canal and aquifer heads increased slightly may have indicated the limited ability of the Biscayne aquifer to respond to large volumes of water released through structures or that walls of the canals in recharge reaches became clogged. For construction of a regional flow model of the study area, canal stages were identified with the local water-table altitude.

A numerical simulation was attempted to solve the many special problems of regional simulation of flows in the Biscayne aquifer by modifying an existing computer code and using innovative application techniques. The code selected for the simulation (SWIP) is a three-dimensional finite-difference simulator of pressure and solute and thermal transport. Absolute pressure is the solution variable of the flow equation. The model simulates a free surface by comparing pressures in the uppermost layer of grid cells with a user-specified base pressure to determine the saturated thickness of the cells. For this study, the saturated thickness test was extended to cells in lower layers, and an approximation was introduced to permit rewetting of dry cells.

In southern Florida, the response of the water table to seasonal changes in rates of recharge and evapotranspiration losses is the most important facet of aquifer behavior. For this reason, a transient simulation was attempted that replicated the seasonal behavior of the water table. This required a procedure for simulating the influence of the seasonal body of surface water on the ground-water flow system, both as a control on ground-water heads and as a source or sink of water flux. In the selected approach, the upper layer of the model represented overland surface flow. Because no flow data from the Everglades were available, the surface-flow computations could not be verified. However, use of the model as a simulator of flows in the Biscayne aquifer merely required the simulation of stages in the surface-flow region.

Strings of low-permeability cells in the overland flow layer represented levees that act as a barrier to surface flow. Canals were represented by strings of narrow cells in parts of the next underlying layer of the model grid. High equivalent hydraulic conductivity coefficients (10^8 to 10^9 ft/d) were assigned to these cells. Inherent in this treatment was the assumption that canals were in direct hydraulic connection with the aquifer. Most of the canals were represented as being 100 ft wide and 15 ft deep. Control structures and their operation were represented by specifying control elevations at selected cells within the canal cell strings.

Model boundaries were selected to correspond to locations of existing surface-water stage and well water-level data collection sites, both along the coasts of Biscayne and Florida Bays and inland. A subroutine was coded that processes a time history of heads from selected locations along model boundaries and assigns current hydrostatic pressure values to boundary faces. Along the bay coasts, seawater density was used for the hydrostatic pressure assignments. Other time-varying parameter specifications (rainfall rates, maximum evapotranspiration rates, pumping rates, control-structure elevations, and coastal discharges) were also entered using the new subroutine.

Separate models were constructed for the five water-management time periods (water years 1945-52, 1953-61, 1962-67, 1968-82, and 1983-89) representing stages in the development of the system of canals, levees, and control structures. Calibration of heads in the first two periods was mainly based on adjustment of parameters representing natural properties of the aquifer and surface-water flow system. These parameters were unchanged in later time periods when the representations of new water-management features were tested. Generally, calibration of the model in early time periods was by assigning effective porosity values and monthly rainfall totals and by adjusting aquifer hydraulic conductivities, the equivalent hydraulic conductivity of overland flow, maximum evapotranspiration rates, and shallow- and deep-root zone depths. Some adjustments were made to individual monthly rainfall rates when previously specified values appeared to be unrepresentative of the general area.

An equivalent hydraulic conductivity of 3,000,000 ft/d was assigned to the overland flow layer, except in Taylor Slough where a value of 100,000,000 ft/d was used. The peat and marly soil layers were represented by assigning a hydraulic conductivity value of 10 ft/d to sections of layer 2. Layer 3, representing the Miami Limestone, and layer 4, representing the Fort Thompson Formation and contemporaneous deposits, were assigned a hydraulic conductivity value of 30,000 ft/d, except for the low-permeability section in the northwest and a section in the northeast where a lower value of 5,000 ft/d was assigned. A spatially uniform porosity value of

20 percent was assigned to layers 2, 3, and 4. Calibrated maximum evapotranspiration rates ranged from a January low of 0.08 in/d (inch per day) to a summer high of 0.21 in/d.

The results of sensitivity analyses and water-budget analyses illustrated the behavior of the flow system by identifying the processes functioning as primary natural controls on the water-table altitude. The model appeared to be relatively insensitive to moderate changes in values of hydraulic conductivity and porosity. When the equivalent hydraulic conductivity assigned to overland sheetflow was increased and decreased by 50 percent, the effect on stages was slight. Rainfall and maximum evapotranspiration rates were each varied by 20 percent with a substantial effect on ground-water heads and surface-water stages. Because measured summer rainfall totals can easily have local variations of several inches, the need to correlate and verify summer rainfall data among several measuring stations was apparent.

Flow processes in the modeled region were identified by water mass flux analyses, in which a quantitative description of lateral and vertical mass fluxes in vertical columns of unit width and length was obtained from the model output from selected times in the 1953 water year. Results showed that the net horizontal mass flux (net loss or gain from aquifer flow in the vertical column) was small compared to the average flux and was usually relatively small compared to the net atmospheric flux (recharge minus evapotranspiration), explaining why the atmospheric recharge and loss parameters influence the water table so strongly.

In the first water-management time period (water years 1945-52), a large region of the southwestern part of the county was simulated as having water-table altitudes below those near the coast during an average dry period. During an average high-water period, a high water mound was simulated in the north-central part of the study area. Only the high parts of the coastal ridge were depicted as unaffected by flooding in 1947. During high-water periods, simulated aquifer flows toward the east and south were highest near U.S. 1 and Ingraham Highway in Everglades National Park. During recession periods the region of greatest simulated aquifer flows shifted gradually inland, and in low water periods the highest flows occurred in the north-central region where the high water mound had previously occurred. During the 1945-52 time period, the ratio of evapotranspiration to rainfall recharge was computed by the model to be 88.3 percent. The significant hydraulic influence of the agricultural drainage canals extending to the coast of lower Biscayne Bay during the 1953-61 water-management time period was demonstrated using a sensitivity analysis approach in which the representation of agricultural drainage was entirely deleted. During the 1953-61 time period, the ratio of evapotranspiration to rainfall recharge was computed to be 94.0 percent. The simulation of drainage by Black Creek Canal in the 1962-67 water-management time period depicts substantial lowering of the water table locally and appreciable lowering of the water table throughout most of the noninundated northeastern part of the study area. The presence of the canal caused stronger and earlier aquifer drainage south of Tamiami Trail and west of L-31N. During the 1962-67 time period, the ratio of evapotranspiration to rainfall recharge was computed to be 93.3 percent.

The principal features of the extensive flood-control system of levees, canals, and control structures that went into operation at the beginning of the fourth water-management time period (water years 1968-82) were represented in the model design. After major storms, computed recessions at many data sites in the region of southern Dade County drained by the flood-control system were too long in duration compared with actual recessions, because the monthly averaging of inputs representing rainfall and control structure operation did not permit the simulation of the rapid operation of the water-management system for flood control. During the 1968-82 time period, the ratio of evapotranspiration to rainfall recharge was computed to be 90.6 percent. A sensitivity analysis was made to determine the simulated influence of the flood-control system in the 1968-82 water-management time period. In the analysis, the post-1967 canal system was deleted, except that L-67 Extended and canal continued to be represented. The principal effect on the water table in the region drained by the flood-control system was to raise and prolong average peak water-table altitudes by a modest amount. A second sensitivity analysis was designed to assess the influence of L-67 Extended and canal. The general result of deleting the canal and levee representations was an eastward spreading and shallowing of surface-water flows released through the S-12 structures.

Calibration of the flow model for the fifth water-management time period (water years 1983-89) required that the modeled region be divided into two rainfall regions, differing in specified summer rainfall totals for 1985, 1986, and 1987. Simulation required representation of the operation of the S-331 pumping station in the L-31N canal and the nearly permanent openings of most control structures in the southern coastal ridge. The influence of the South Dade Conveyance System was assessed by deleting the S-331 pumping and the continuous structure openings to restore the 1968-82 flood-control system. Results of the sensitivity analysis were a general lowering of the water table in the southern coastal ridge of 0.5 to 1.5 ft. Results of the simulation indicated that the conveyance system and the S-331 pumping raised the water table in that region by forcing large quantities of water into the local canal system. The pumped water then recharged the aquifer as the water flowed through the canals toward coastal controls.

During time period 5, the ratio of evapotranspiration to rainfall recharge was computed to be 97.9 percent--higher than in earlier water-management time periods. A comparison of average simulated rainfall and evapotranspiration rates from all time periods showed average rainfall for the 1983-89 water years to be less than that of any other time period, and the average evapotranspiration rate to be higher than that in every time period except the 1953-61 time period. Raising water levels in the southern coastal ridge by importation of water from the north had the effect of increasing evapotranspiration, even though the average rate of rainfall was low.

Further insight into the behavior of the model and into the hydrology of the study area was gained by representing generic well-field pumping scenarios in the fifth time-period simulation, in which conditions were similar to those that would prevail should pumping occur. Results of the analyses suggested that, depending on the location of the well field with respect to the canal, as much as 80 percent of the pumped water could be supplied by the canal, assuming that canal stages were maintained by supplying equal amounts of water at the northern boundary. The remaining water was taken from storage in the Biscayne aquifer.

The techniques employed in this study to represent the influences of overland sheetflow, rewetting, canal flows, and control-structure operation should have transfer value to other regions similar to the study area. They may even be applicable when canals are completed in materials less permeable than the underlying aquifer, provided that it can be established that little canal wall clogging occurs that would confine the canal waters. The accuracy with which water-table changes influenced by canal drainage are simulated could be improved by providing for daily parameter input specifications, although this would require lengthier computations. Although the modeling techniques employed in this study are not intended to be used to simulate surface-water flow rates, such a capability could be gained by the development and implementation of more sophisticated algorithms for surface-water flow rate computation and by obtaining field measurements for calibration.

The water-management time-period simulations developed for this study might have further potential use for the type of general regional evaluations and sensitivity analyses that have been described in this report if care is taken that input specifications (boundary conditions, coastal canal discharges, rainfall rates, and control structure operational criteria) are consistent with the design of the new analyses. The generalized and regional nature of the five water-management time-period models cautions against using them for detailed local interpretations. Where heads are accurately simulated in the regional-scale model, they can be used to provide boundary specifications for subregional models that use the same simulation techniques.

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