Description and Field Analysis of a Coupled Ground-Water/Surface-Water Flow Model (MODFLOW/BRANCH) with Modifications for Structures and Wetlands in Southern Dade County, Florida

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Abstract

A coupled surface-water model (BRANCH) and ground-water model (MODFLOW) model were tested to simulate the interacting wetlands/surface-water/ground-water system of southern Dade County. Several options created for the MODFLOW ground-water model were used in representing this field situation. The primary option is the MODBRANCH interfacing software, which allows leakage to be accounted for between the MODFLOW ground-water model and the BRANCH dynamic model for simulation of flow in an interconnected network of open channels. A modification to an existing software routine, which is referred to as BCF2, allows cells in MODFLOW to rewet when dry—a requirement in representing the seasonal wetlands in Dade County. A companion to BCF2 is the modified evapotranspiration routine EVT2. The EVT2 routine changes the cells where evapotranspiration occurs, depending on which cells are wet. The Streamlink package represents direct connections between the canals and wetlands at locations where canals open directly into overland flow. Within the BRANCH model, the capability to represent the numerous hydraulic structures, gated spillways, gated culverts, and pumps was added.

The application of these modifications to model surface-water/ground-water interactions in southern Dade County demonstrated the usefulness of the coupled MODFLOW/BRANCH model. Ground-water and surface-water flows are both simulated with dynamic models. Flow exchange between models, intermittent wetting and drying, evapotranspiration, and hydraulic structure operations are all represented appropriately. Comparison was made with a simulation using the RIV1 package instead of MOD-BRANCH to represent the canals. RIV1 represents the canals by user-defined stages and computes leakage to the aquifer. Greater accuracy in reproducing measured ground-water heads was achieved with MODBRANCH, which also computes dynamic flow conditions in the canals, unlike RIV1.

The surface-water integrated flow and transport two-dimensional model (SWIFT2D) was also applied to the southeastern coastal wetlands for comparison with the wetlands flow approximation made in MODFLOW. MODFLOW simulates the wetlands as a highly conductive upper layer of the aquifer, whereas SWIFT2D solves the hydrodynamic equations. Comparison in this limited test demonstrated no specific advantage for either method of representation. However, much additional testing on a wider variety of geometric and hydraulic situations, such as in areas with greater tidal or other dynamic forcing effects, is needed to make definite conclusions.

A submodel of the existing southern Dade County model schematization was used to examine water-delivery alternatives proposed by the U.S. Army Corps of Engineers. For this application, the coupled MODFLOW/BRANCH model

was used as a design tool. A new canal and several pumps to be tested to maintain lower water levels in a residential area (while water levels in the Everglades are raised) were added to the model schematization. The pumps were assumed to have infinite supply capacity in the model so that their maximum pumping rates during the simulation could be used to determine pump sizes.

INTRODUCTION

Southern Florida is an environmentally and hydrologically unique area because of the wide and shallow wetlands known as the Everglades, the living coral reef off its shoreline, its highly porous substrata, and its subtropical marine climate. Federal, State, and local water officials have expressed concern over the environmental deterioration of the area due to changes in the natural flow system and chemical changes to ground water and surface water resulting from the area's burgeoning population. Problems that water managers are currently addressing include the lowering of water levels and alteration of the hydroperiod (or timing of wet and dry periods) in the Everglades and Florida Bay, saltwater intrusion into aquifers, and chemical contamination of ground and surface waters from domestic, commercial, industrial, and agricultural activities.

Mathematical modeling techniques are used to study these problems in southern Florida. Separate models have been designed to study wetlands, surface water, and ground water. However, no one type of flow model has been effective in simulating the system as a whole. This is because the unsaturated zone and the shallow water-table aquifer are extremely porous, and the seasonal and tidal wetlands, aquifer, and numerous canals within the area are all in direct hydraulic connection.

The U.S. Geological Survey (USGS) has developed a widely used, modular, ground-water flow model known as MODFLOW (McDonald and Harbaugh, 1988). Two recent modifications to MODFLOW have facilitated modeling the southern Florida hydrologic system. The first modification is the design of a module called MODBRANCH (Swain and Wexler, 1993), which allows MODFLOW to be linked to a dynamic model of flow in surface-water networks known as BRANCH (Schaffranek and others, 1981; Schaffranek, 1987). The second modification to MODFLOW is development of a new version of the BCF package, BCF2 (McDonald and others, 1991), also known as the wetdry package, which allows model cells to alternate between wet and dry states, or active and inactive states in modeling parlance. This modification, with the judicious use of package options, permits the seasonal and tidal wetlands to be represented as a layer in a ground-water flow model. The original and modified versions of the BCF package are documented in McDonald and Harbaugh (1988) and McDonald and others (1991), respectively. Models (defined as stand-alone programs) and model packages (defined as subroutines used by a model for a specific purpose) referenced in this report are described in table 1.

Once the major obstacles of simulating the ground-water/surface-water interface and representing wetlands in a ground-water flow model had been overcome, there were several other problems that needed to be addressed, some of which are unique to southern Florida. These problems included:

- In southern Florida, wetlands are in direct hydraulic connection with the aquifer. A method of representing these wetlands in a ground-water flow model needed to be developed.
- MODFLOW simulates evapotranspiration losses from the uppermost model layer cells or from cells designated at the beginning of each stress period. When BCF2 is used to represent a seasonal wetlands, evapotranspiration losses should be subtracted from the uppermost wet cells.
- In southern Florida, specifically Dade County, canals deliver water directly into wetlands areas. Although river and ground-water flow could be simulated conjunctively using MODBRANCH, a model technique was needed to link the river flow model directly to wetlands areas. A code for accomplishing this task (Streamlink) was developed as part of this study and is documented in Swain (1993).
- Surface-water (canal) flow in southern Florida is strictly controlled by pumps, gates, and culverts operated by the South Florida Water Management District. These structures need to be accurately represented in an integrated ground-water/surface-water flow model by incorporating algorithms that represent discharge rating curves. A technique for accomplishing this originally was developed as part of this study and is documented in Swain (1992).

Table 1. Models and model packages referenced in report

Model or model package name	Type	Description	Reference
MODFLOW	Model	Represents three-dimensional ground-water flow	McDonald and Harbaugh (1988)
BRANCH	Model	Represents one-dimensional dynamic streamflow	Schaffranek and others (1981) Schaffranek (1987)
MODBRANCH	Model Package	Links MODFLOW to BRANCH to account for stream-aquifer leakage	Swain and Wexler (1993)
BCF2	Model Package	Revised computational package allowing rewetting of dry cells in MODFLOW	McDonald and others (1991)
RIV1 (River Package)	Model Package	Package to represent rivers in MODFLOW with user-defined stages	McDonald and Harbaugh (1988)
STR1 (Stream Package)	Model Package	Package to represent rivers in MODFLOW with stream-routing algorithm	Prudic (1989)
Streamlink	Model Package	Package to directly assign water level and flow values between RIV1, STR1, MODBRANCH and MODFLOW	Swain (1993)
EVT2	Model Package	Revised computational package allowing model cells from which evapotranspiration is removed to change based on wetting and drying in MODFLOW	This report
Structures subroutine in BRANCH	Model Package	Allows representation of hydraulic control structures in BRANCH	Swain (1992)
SWIFT2D	Model	Represents two-dimensional dynamic surface-water flow	Leendertse (1987) Regan and Schaffranek (1993)

- Dade County has tidal wetlands along its southern and southeastern boundaries that need to be simulated in models of the area. Two alternatives for doing so, the aquifer block method, MODFLOW, and the use of a surface-water model, SWIFT2D (Regan and Schaffranek, 1993), needed to be compared to determine their relative merits.

To address these problems, the USGS, in cooperation with the South Florida Water Management District (SFWMD), conducted a study to develop methods for simulating the integrated wetlands/surface-water/ground-water systems in southern Dade County. Dade County was selected because it includes within its boundaries all of the conditions that need to be considered in an integrated hydrologic model of southern Florida. MODBRANCH, Streamlink, and the algorithms for representing control structures were developed independently. In addition to addressing the representation of wetlands, it was uncertain if these computationally intensive programs were feasible on a regional scale and if their use would simulate a wetlands/surface-water/ground-water system better than the nonintegrated models previously used.

Purpose and Scope

The purpose of this report is to describe the coupling of a ground-water and surface-water flow model (MODFLOW/BRANCH) and present the results of a field-scale application of the coupled flow model, modified to simulate control structures and wetlands in southern Dade County. The coupled model was developed to test techniques for simulating open-channel and overland flows in conjunction with the MODFLOW ground-water flow model. This report describes: (1) modifications to the MODFLOW and BRANCH models; (2) development of algorithms that incorporate hydraulic structures into the BRANCH model; (3) construction, calibration, and verification of the coupled MODFLOW/BRANCH model; and (4) comparisons of methods that interface wetlands to the coupled ground-water/surface-water models.

The modifications made to the MODFLOW and BRANCH models allowed representation of the hydraulically well-connected wetlands-canal-aquifer systems of southern Dade County. A subroutine was developed for BRANCH to represent the hydraulic structures that regulate the canal stages in Dade County. A comparison was made of model results with canals represented by MODBRANCH in contrast with canals represented by the River package (McDonald and Harbaugh, 1988). Additionally, canal representations using the SWIFT2D surface-water model and the MODFLOW/MODBRANCH model were compared. Finally, a submodel, representing an area known as the 8.5-mi^2 (square mile) residential area, was used to explore water-delivery alternatives to Everglades National Park and associated areas proposed by the U.S. Army Corps of Engineers (COE). The final model developed as part of this study was designed to test algorithms and techniques for representing the interaction between wetlands, surface water, and ground water. The model is not intended to be a management model of southern Dade County.

Acknowledgments

The authors wish to express appreciation to their USGS colleague, Michael Merritt, whose existing Subsurface Waste Injection Program (SWIP) model was used to generate aquifer parameters and geometry. Mark Wilsnack, an engineer with the South Florida Water Management District, was instrumental in compiling attribute data on the primary canals represented in the model.

DESCRIPTION OF STUDY AREA

Southern Dade County, defined as the area south of the Tamiami Canal, is located on the southeastern tip of Florida (fig. 1). A topographic high, the Atlantic Coastal Ridge (fig. 2), separates the coastal marshland and mangrove swamps from the Everglades to the west. The Atlantic Coastal Ridge is 2 to 8 mi (miles) wide with land-surface altitudes ranging from 8 to 12 ft (feet) above sea level. The Atlantic Coastal Ridge is a natural barrier to drainage from the interior, except where the ridge is transversed by canals or natural sloughs. Coastward from the Atlantic Coastal Ridge, the marshland and mangrove swamps range in altitude from 0 to 3 ft above sea level. Parts of this area are covered by tidal wetlands. West of the Atlantic Coastal Ridge, the altitude of the Everglades ranges from 1 to 9 ft above sea level. The part of the original historical Everglades east of levees L-31N and L-31W (fig. 1) has been drained for development and agriculture, although there are small remnant wetlands just east of levee L-31N between Tamiami Canal and Bird Drive Canal (fig. 1). The area west of the levees is primarily wetlands, a large part of which lies within the boundary of Everglades National Park (ENP).

Natural surface drainage in southern Florida has been greatly reduced and its direction altered by the construction of a network of controlled drainage canals. Historically, the area east and south of the Atlantic Coastal Ridge flowed toward the coast, and the area west and north of the ridge flowed toward the south and southwest. These drainage patterns have been somewhat altered east of the L-31 system where the limited surface flows that do occur are toward the canals (except for the near-coastal areas).

Vertical drainage or infiltration rates vary according to soil covering (fig. 2). East of the Atlantic Coastal Ridge, the underlying limestone is covered with several inches to several feet of the Perrine Marl, characterized by poor to moderate vertical permeability. The Atlantic Coastal Ridge and part of the Everglades (the Rocky Glades) have a Rockdale soil or Rockland covering. These are not true soils but consist of soft limestone with varying amounts of fine sand or fine sandy loam. Vertical permeability is moderately high. The northwestern part of the Everglades in the study area is covered by several feet of peat or marl, characterized by poor natural drainage. A more detailed discussion of Dade County soils and soil properties is given by the U.S. Department of Agriculture (1958) and Caldwell and Johnson (1982).

Hydrogeology

Most of the study area is underlain by the Biscayne aquifer, defined by Fish and Stewart (1991, p. 12) as follows: "That part of the surficial aquifer in southeastern Florida composed of (from land surface downward): the Pamlico Sand, Miami Limestone [Oolite], Anastasia Formation, Key Largo Limestone, and Fort Thompson Formation (all of Pleistocene age) and contiguous, highly permeable beds of the Tamiami Formation of Pliocene and late Miocene age where at least 10 ft of the section is very highly permeable (a lateral [horizontal] hydraulic conductivity of about 1,000 ft/d [feet per day] or more)." The Biscayne aquifer is a shallow water-table aquifer that extends from near land surface to depths of between 20 and 120 ft

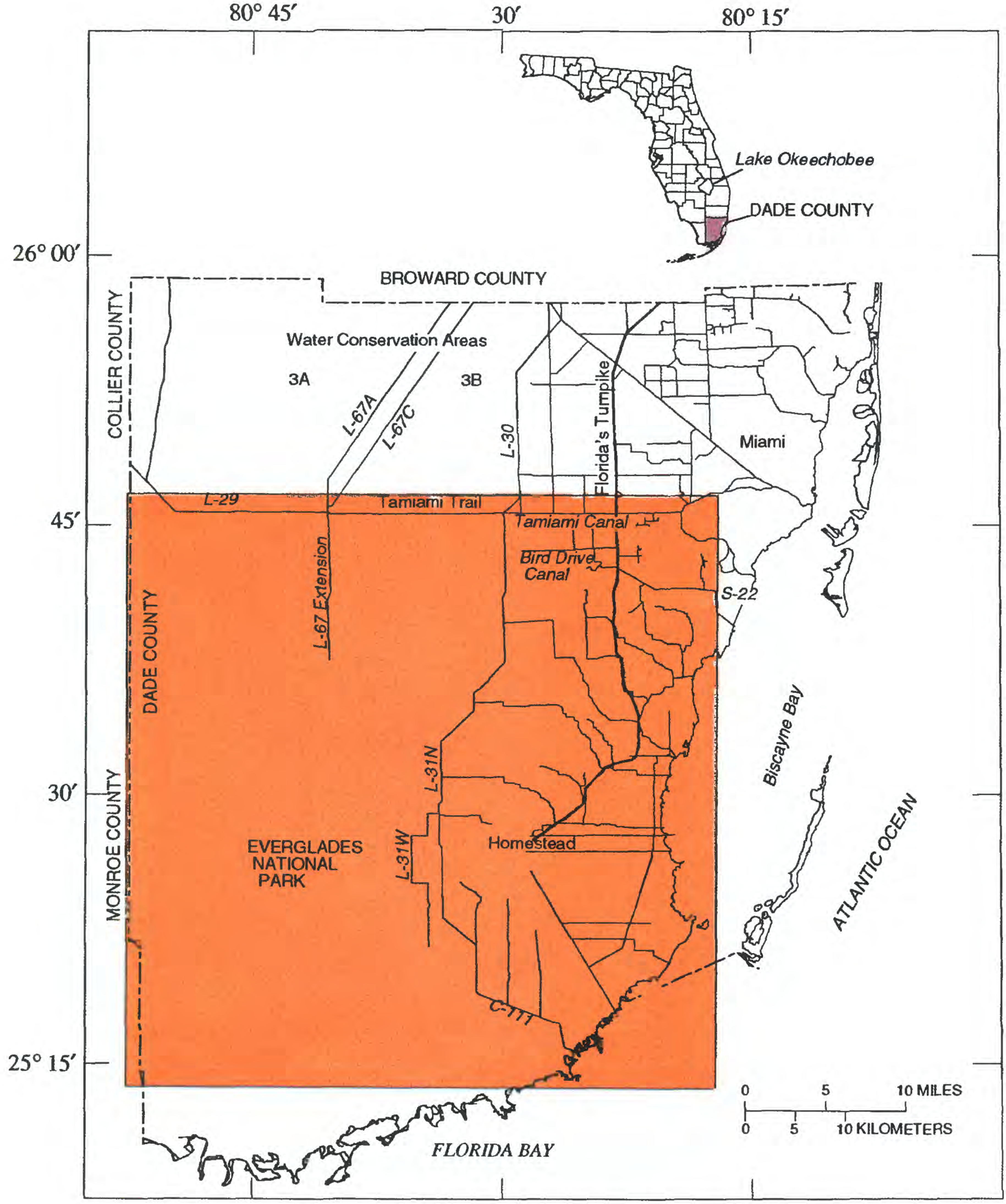

Figure 1. Location of study area showing water-management canals.

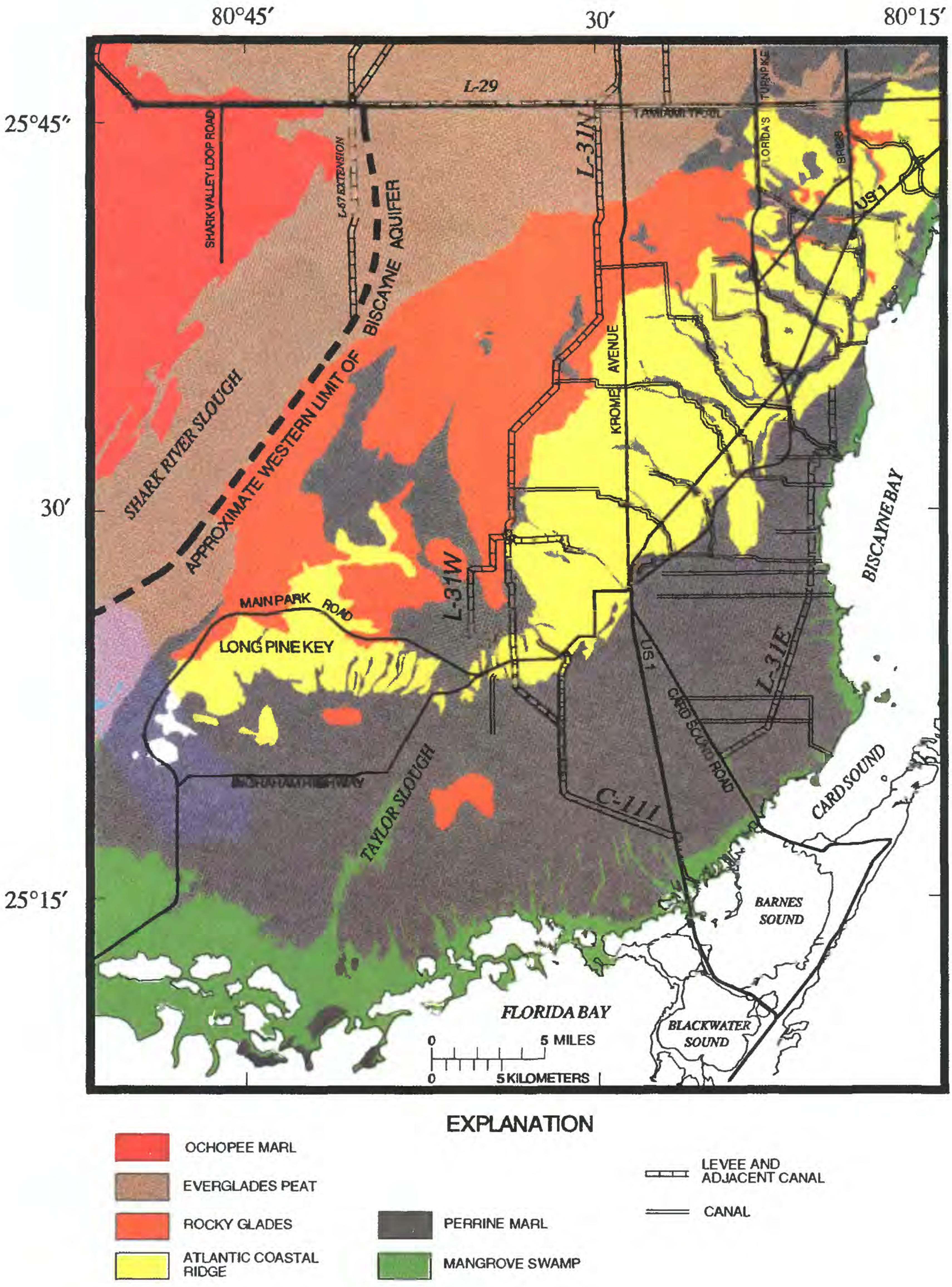

Figure 2. Soil and physiographic features of the study area (modified from Merritt, 1995).

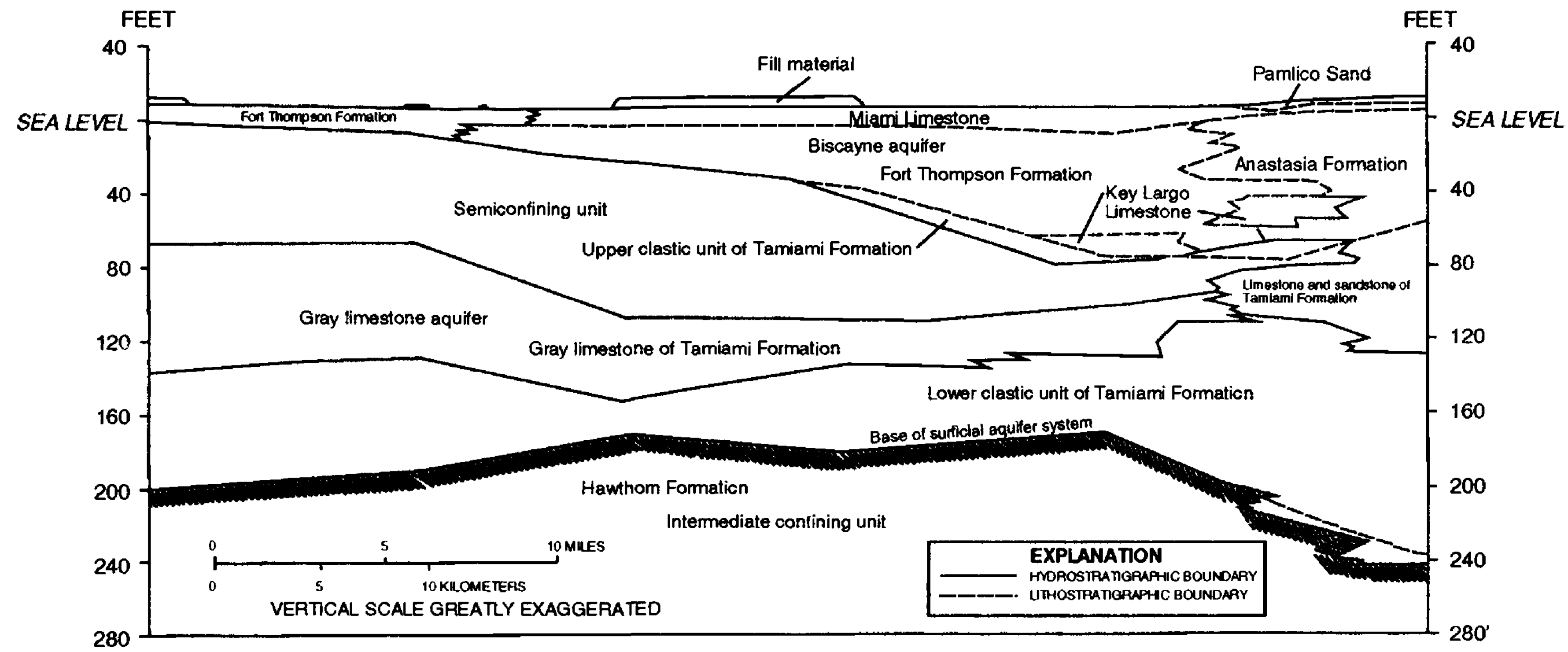

Figure 3. Schematic relations of geologic formations and aquifers within the surficial aquifer system (modified from Fish and Stewart, 1991, p. 11).

below sea level along the east coast in the study area. The aquifer pinches out in the west and is not present in the northwestern part of the study area where a less permeable facies of the Fort Thompson Formation is present (figs. 2 and 3). Below the Biscayne aquifer, the upper part of the Tamiami Formation is less permeable and acts as a semiconfining unit under all but the northeastern part of the study area. Figure 3 shows the relation of the Biscayne aquifer and geologic formations in the study area, the surficial aquifer, and the underlying confining Hawthorn Formation.

Surface deposits below the study area are primarily Miami Limestone (formerly called Miami Oolite) or a few feet of peat, muck, or limemud overlying the Miami Limestone. The peat and muck are organic deposits generally present in the northwestern part of the study area or in the southern coastal areas. Lateral hydraulic conductivity data for the peat and muck are limited; however, the unit is considered to be of low permeability. Limemud is mainly in the southwestern coastal parts of the study area and is considered to be relatively impermeable by Fish and Stewart (1991, p. 27). The peat, muck, and limemud sediments were assigned lateral hydraulic conductivities ranging from less than 0.1 to 100 ft/d by Fish and Stewart (1991).

Miami Limestone either underlies the peat, muck, and limemud or is present at the surface in virtually all of the study area. It was present in all of the test holes previously drilled in the study area by Fish and Stewart (1991), except for one well in the extreme northwestern part of the study area and another in the extreme southeastern part. Miami Limestone thickens toward the east and south, and its thickness varies from a few feet in the northwestern part of the study area to more than 30 ft beneath the Atlantic Coastal Ridge. Lateral hydraulic conductivity varies, depending on the degree of cementation and the development of secondary porosity. The limestone tends to be less permeable in the northwestern part of the study area where lateral hydraulic conductivity ranges from 0.1 to 10 ft/d and is more permeable in the eastern and southern parts where lateral hydraulic conductivity is greater than 1,000 ft/d (Fish and Stewart, 1991).

The Fort Thompson Formation underlies the Miami Limestone throughout most of the study area and occurs at the surface in the northwestern part where the Miami Limestone is absent (fig. 3). The Fort Thompson Formation is less than 10 ft thick in the northwestern part of the study area, but thickens to the south and east, reaching a maximum thickness of 60 ft in the northeastern part in the vicinity of Florida's Turnpike (fig. 1). This unit is composed of marine and freshwater limestones, with lateral hydraulic conductivities as low as 0.1 to 10 ft/d in the far northwestern part of the study area. However, below most of the study area, lateral hydraulic conductivity is greater than 1,000 ft/d according to Fish and Stewart (1991), who suggest that conductivities might average more than 40,000 ft/d in the western half of the study area.

The Anastasia Formation (and not the Fort Thompson Formation) underlies the Miami Limestone in the northeastern part of the study area. This unit is thickest (80 ft) in coastal areas in the vicinity of the Tamiami Canal and thins to the west and south as it interfingers with the Fort Thompson Formation and Key Largo Limestone. The Anastasia Formation is composed of shelly sandstone and limestone interbedded with sand and is usually less permeable than the Fort Thompson Formation, with lateral hydraulic conductivities ranging from 10 to more than 1,000 ft/d (Fish and Stewart, 1991).

The Key Largo Limestone is not widespread in the study area. This unit occurs at the surface in the far southeastern part of the study area (Fish and Stewart, 1991). In coastal areas, the Key Largo Limestone is present as thin zones, interfingering with the Fort Thompson Formation and Anastasia Formation. Fish and Stewart (1991, p. 33) estimated that the lateral hydraulic conductivity of the Fort Thompson Formation, Anastasia Formation, and Key Largo Limestone (taken as a unit) is greater than 10,000 ft/d.

In some parts of the study area, the more permeable upper part of the Tamiami Formation is included in the Biscayne aquifer. Inclusion of the upper part of the Tamiami Formation is most predominant in the northeast where it might be as much as 60 ft thick. The limestones, calcareous sandstones, and sand of the Tamiami Formation are not as permeable as the Fort Thompson Formation, but lateral hydraulic conductivity might still exceed 1,000 ft/d (Fish and Stewart, 1991).

The semiconfining unit that underlies the Biscayne aquifer is composed of sand, silt, clay, shell, sandstone, limestone, and organic sediments assigned to the upper Tamiami Formation (Fish and Stewart, 1991; Causaras, 1987). In the far northeastern part of the study area, the semiconfining unit of the upper Tamiami Formation does not occur, and the Biscayne aquifer is confined by the lower Tamiami Formation. In the rest of the study area, the semiconfining unit increases in thickness in a westerly and southerly direction, ranging in thickness from less than 15 ft, about 7.5 mi west of Florida's Turnpike and the Tamiami Canal (fig. 1), to more than 130 ft in the vicinity of Homestead. The lateral hydraulic conductivity of the semiconfining unit usually ranges from 0.1 to 100 ft/d, although there are noncontinuous lenses of less permeable sediments interbedded in the unit.

Canal and Levee System

Southern Florida is traversed by a series of canals and levees, many built by the COE, and maintained and operated by the SFWMD. Canals built by the COE are usually assigned a C number (for example, C-1 or C-102). Branches of these canals may have a locational suffix (for example, C-103S for the southern branch of C-103). Levees are assigned an L number (for example, L-31 or L-67 Extension). The seepage canal adjacent to a levee is usually referred to by the levee number (for example, L-31 Canal or L-67 Extension Canal). An exception to this protocol is the C-111 levee, in which the levee is referred to by the canal number. Most major canals also have names, such as Tamiami Canal or Black Creek Canal. Pumps and structures on canals or in levees are usually referred to with an S number (for example, S-173); however, a few have a G designation (for example, G-93).

Primary canals carry surface water from Lake Okeechobee in central Florida to coastal areas where the water replenishes ground-water supplies and provides the hydrostatic pressure to prevent saltwater intrusion. During times of heavy rainfall, these canals can be used to provide flood control by draining the coastal areas. Secondary canals connect to the primary system and provide the same functions to smaller localized areas. In some cases, smaller tertiary canals, operated by counties or local drainage districts, connect to secondary canals to drain or replenish even smaller areas.

Surface water moves through the system by a series of pumps and control structures (gated culverts and spillways), each having its own operating rules. Operating rules govern how a structure works much of the time. The operating rules may be based on one of several criteria: the stage gradient between upstream and downstream sides of the structure, the stage of some point upstream or downstream of the structure, or in rare instances, the water level at a ground-water recording station. Operating rules are written so that when a criterion for the structure is met (for example, the upstream stage reaches a specified elevation), the pump is activated at a specified rate or the gate opens to a specified position. The pump stays on or the gate remains open until a second criterion is met (for example, the downstream stage reaches a specified elevation). At this time, the pump shuts off or the gate returns to its normal operating position. However, it is not unusual for a structure to be operated differently from the normal operating rules. This is usually done for pragmatic reasons, such as the release of excessive stormwater runoff, lowering stages in anticipation of tropical storms, or manatee migrations which preclude the closing of control gates.

A system of levees works in connection with the canal system to control surface-water flow. From Lake Okeechobee south, the western levee system divides the higher water levels of the Everglades wetlands from the lower water levels in populated coastal areas. Seepage under the levees is captured by canals where it adds to the coastward flow. North of the study area, levees are used to impound surface water in water-conservation areas, from which it can be released into canals by direct connection or by seepage. All of the surface water in the study area, other than rainfall recharge, originates from the western canal-levee system or as controlled releases or seepage from conservation areas.

Surface-water features and levees in the study area are shown in figure 4. The L-31N/C-111 levee system divides the area into two general water-management regimes. West of the levee system, the primary water-management objective is to provide water to the ENP drainage system, which includes Shark River Slough and Taylor Slough (fig. 2). East of the levee system, water management provides recharge to the coastal aquifer, flood control, and a barrier to saltwater intrusion.

Water enters the ENP drainage basin from the north by discharge through the S-12 control gates between L-67 Extension and Forty-Mile Bend, and from culverts under the Tamiami Trail (US-41) between L-31N and L-67 Extension. During the period selected for flow analysis (1990), the weekly discharges through these structures were controlled based on the preceding weekly rainfall, evapotranspiration, and stages in Water Conservation Area 3A (Cooper and Roy, 1991). Discharges into this area of the ENP drainage basin are controlled by the operation of the S-12 structures and the coordinated operation of S-333 and S-334 on the L-29 Canal, which forms the northern boundary of the study area.

The L-67 Extension levee and canal divide the headwaters of Shark River Slough into eastern and western sections. The L-67 Extension levee and canal were originally intended to deliver more water into Shark River Slough; however, they had the effect of short circuiting the natural sheetflow in Shark River Slough and of reducing water levels in Northeast Shark River Slough. As a result, the two culverts, S-346 and S-347 (not shown), in the L-67 Extension Canal are left closed.

During the study period, regulations required that the SFWMD deliver discharges of 55,000 acre-ft/yr (acre-feet per year) to the southeastern part of the ENP drainage basin. This was accomplished by a pumping station (S-332) and gated culvert (S-175) in the L-31W Canal which supply water to Taylor Slough and by gaps in the C-111 levee between S-18C and S-197.

Unlike the area west of the L-31N levee system, the area to the east is divided into several drainage basins. The Tamiami Canal (C-4) on the northern boundary of the study area receives flow from the L-30 Canal (fig. 4) to the north through S-335 (not shown). The eastward flowing Tamiami Canal furnishes water through open-channel connections to the Snapper Creek Canal (C-2) and the Coral Gables Canal (C-3) in the northeastern part of the study area (fig. 4). The Tamiami Canal connects to the Miami Canal north of the study area. Snapper Creek Canal and Coral Gables Canal both have structures at their termini (S-22 and G-93) to maintain heads and prevent saltwater intrusion during times of low water levels.

The remaining basins east of the western levee can all receive recharge from the L-31N/C-111 Canal system which is part of the South Dade Conveyance System. Flow in the L-31N/C-111 Canal system is regulated by several gated structures (S-173, S-174, S-176, S-177, S-18C, and S-197) and by one pump station (S-331).

All of the remaining canals in the study area drain and/or recharge smaller subbasins by the operation of stage and flow-divide structures (fig. 4). An extensive network of shallow agricultural drainage canals (not shown) exists in the southeastern part of the study area. The coastal levee L-31E (fig. 4) prevents storm surge from inundating the low-lying coastal lands, and its borrow canal receives flows from both the main east-west canals and the smaller drainage canals. The main east-west canals (fig. 4) supplied by the L-31N/C-111 Canal system are Black Creek Canal (C-1), Princeton Canal (C-102), and Mowry Canal (C-103). The Cutler Drain Basin (C-100) is also recharged by the L-31N/C-111 Canal system through its connection to Black Creek Canal at S-122 and

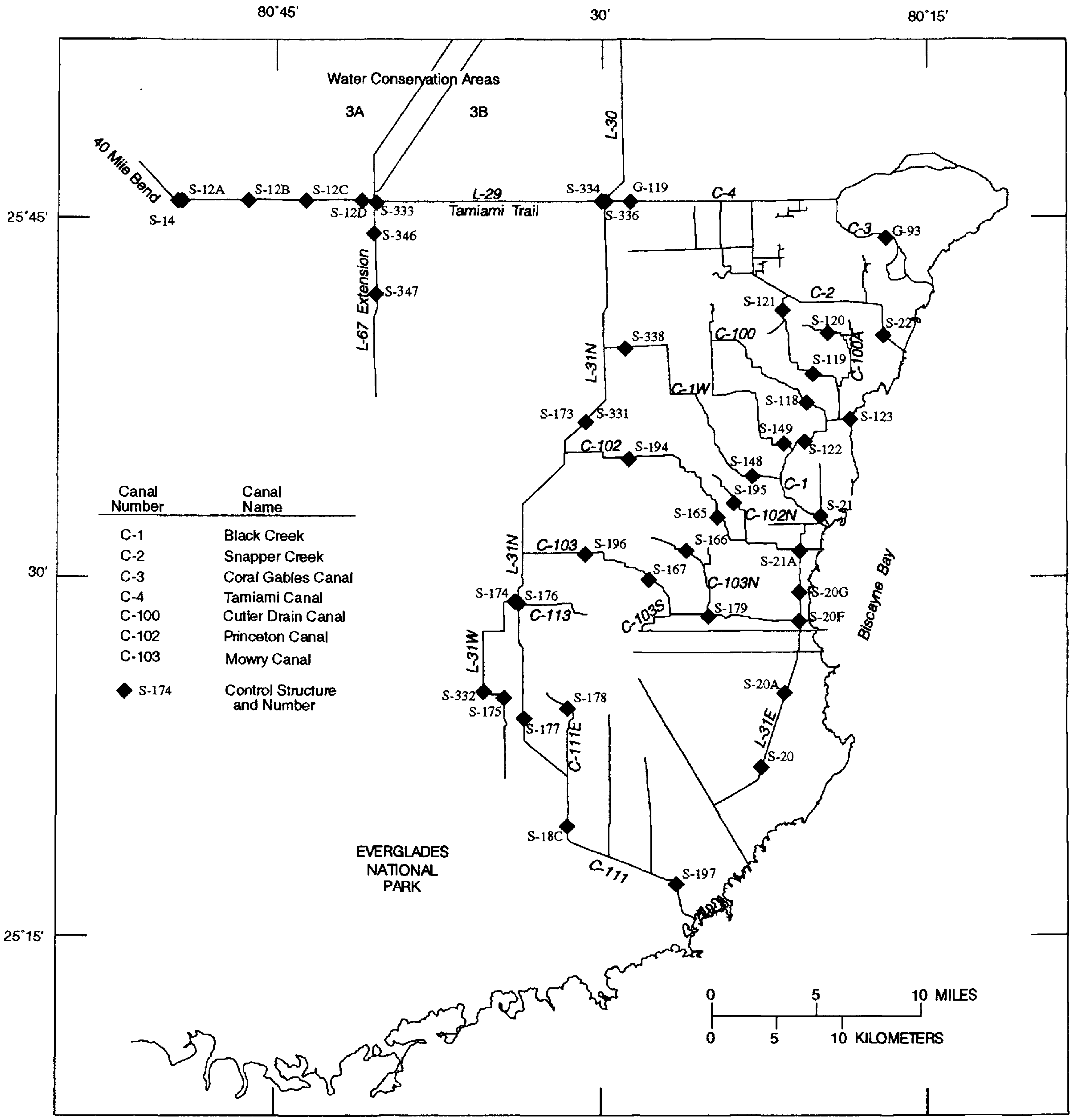

Figure 4. Location of canals, levees, and water-control structures in the study area.

Snapper Creek Canal (C-2) at S-121. Each of the main east-west canals have flow-divide structures (S-338, S-194, and S-196) located east of their open-channel connections to L-31N. Each canal also has salinity control structures (S-21, S-21A, and S-20F) near its terminus at the coastline, as do Cutler Drain Basin (S-123) and Snapper Creek Canal (S-22).

Wetlands

Historically, wetlands constituted the area west of the Atlantic Coastal Ridge. Under current conditions, most of the area west of the L-31 system and the area south of the westward extension of the Atlantic Coastal Ridge are inundated for some part of the year (fig. 4). Within these areas are two deeper sloughs, Shark River Slough and Taylor Slough (fig. 2). Water depths in the southern Dade County wetlands are generally less than 2 ft (Merritt, 1995); however, the depth of water and the acreage inundated vary from year to year depending on the annual rainfall and water-management practices.

Surface-water flow in the wetlands, known as sheetflow, moves very slowly toward the coastal areas because of the low topographic relief. Flow in Shark River Slough is to the southwest away from the study area toward Whitewater Bay (not shown). Flow in the deep Taylor Slough is south, discharging to Florida Bay.

The wetlands are underlain by 1 to 4 ft of peat or marl soils, as previously described. These soils, depending on their thickness, may impede vertical flow, but they do not act as a confining layer. Water levels in the wetlands retreat to below land surface in most of the study area in most years during the dry season, from mid-October to mid-May (Merritt, 1995); water levels then rise above land surface during the wet season, from mid-May to mid-October, as the low-lying ground becomes saturated. Thus, the wetlands and aquifer are in direct hydraulic connection, and one system cannot be studied without considering the other.

Rainfall and Evapotranspiration

Southern Dade County is the only area in the continental United States that is in a subtropical marine climatic regime. Summers are hot and wet, producing about 44 in. (inches) of rainfall, which is near the average annual total (59 in.). Rainfall during the summer wet season (May-October) is usually associated with thunder storms. Rainfall varies widely both in area and in intensity. During the mild, dryer winter months, rainfall is more likely to be associated with cold fronts moving through the area, producing intense rain over wide areas, sometimes for extended periods of time.

Evapotranspiration losses tend to counterbalance recharge to the aquifer from rainfall. The evapotranspiration rate is difficult to measure; however, estimates of maximum evapotranspiration losses as a function of annual rainfall range from 70 percent in urban areas to 95 percent in undeveloped areas (Klein and others, 1975). Evapotranspiration is affected by solar radiation, temperature, land use, ground cover, and depth of the water table, and therefore, like rainfall recharge, rates vary both areally and temporally. Evapotranspiration losses from a water-table aquifer are computed with several assumptions. First, if the water table is above a certain depth, known as the evapotranspiration surface (fig. 5), evapotranspiration losses will occur at a maximum rate based on the previously mentioned factors. Second, if the water table is below the evapotranspiration surface by a certain distance, known as the extinction depth, there will be no losses from evapotranspiration. Finally, if the water table is between the evapotranspiration surface and the extinction depth, evapotranspiration losses will vary linearly from the maximum evapotranspiration rate to 0. In an analysis of hydrographs for the study area, Merritt observed no change in evapotranspiration rate to water-table depths of 11 ft below land surface between the evapotranspiration surface and the extinction depth (M.L. Merritt, U.S. Geological Survey, oral commun., 1993).

DEVELOPMENT OF THE COUPLED MODFLOW/BRANCH MODEL

Several requirements for development of a coupled MODFLOW/BRANCH model are needed to represent the hydrologic regime in southern Dade County. Modifications to the MODFLOW ground-water flow model and the BRANCH surface-water model were required to allow representation of the wetlands-canal-aquifer system of southern Dade County. Additionally, because the surface-water system in Dade County is

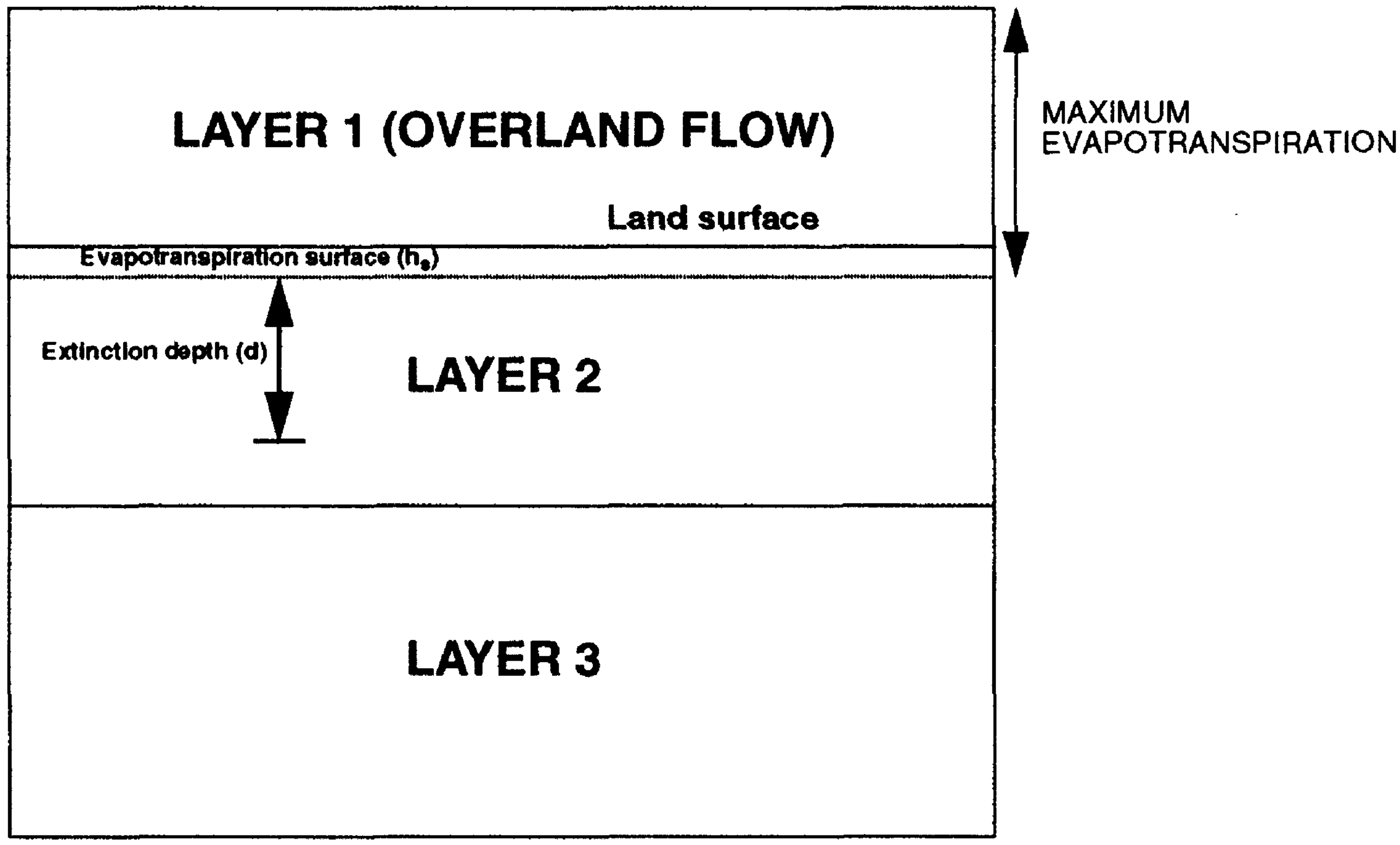

Figure 5. Relation of evapotranspiration to aquifer layers.

highly regulated by hydraulic structures, it was essential that the BRANCH model be capable of representing flow through these structures. Redefining several aquifer parameters used by the MODFLOW model, in terms of the surface-water characteristics of wetlands flow, was also required.

The model was developed to test techniques that more accurately represent surface water in the MODFLOW ground-water flow model. The next sections of this report detail the development of the coupled MODFLOW/BRANCH model. The first section discusses the new modifications that were made to the MODFLOW and BRANCH models, and the second section describes a subroutine that was developed for BRANCH to represent the hydraulic structures that regulate the canal system in Dade County.

Modifications to MODFLOW and BRANCH

As previously discussed, several modifications to the MODFLOW and BRANCH models were required to model the integrated wetlands/surface-water/ground-water system of southern Dade County. Most of the modifications that were needed have been documented in publications by McDonald and others (1991), Swain (1992; 1993), and Swain and Wexler (1993). Those modifications not previously documented and instances where ground-water parameters in MODFLOW are redefined to represent surface-water characteristics are discussed in the subsequent sections.

The Wetdry Package (BCF2)

BCF2 is the package in MODFLOW which calculates the conductance and specific storage[1] in the flow equations that the model solves to determine heads and flow:

$$\frac{\partial}{\partial x}\left(Kxx\ \frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(Kyy\ \frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(Kzz\ \frac{\partial h}{\partial z}\right)-W = S_s\ \frac{\partial h}{\partial t} \tag{1}$$

where,

x is the spatial coordinate in the row direction;

y is the spatial coordinate in the column direction;

z is the spatial coordinate in the layer direction;

Kxx is the lateral hydraulic conductivity in the row direction;

[1]Conductance is defined as the volume of water that flows through a given area of the aquifer under a specified head gradient, and specific storage is defined as the volume of water an aquifer moves from or to storage per unit aquifer volume under a unit change in head.

Kyy is the lateral hydraulic conductivity in the column direction;

Kzz is the lateral hydraulic conductivity in the layer direction;

h is the head at the center of the cell;

W is nonhead-dependent volumetric flux per unit volume into or out of the cell;

S_S is specific storage; and

t is time.

The changes in BCF2, from the original BCF package, which allow model cells to rewet, were developed to simulate the dewatering of an aquifer due to a pumping well. Therefore, using the package to simulate the wetting and drying of a wetlands requires some redefining of BCF2 input parameters, such as lateral hydraulic conductivity and specific storage (which is virtually equal to the effective porosity in a water-table aquifer).

The Kxx, Kyy, and Kzz terms in equation 1 represent the lateral hydraulic conductivity of the aquifer in the x (row), y (column), and z (layer) directions. When the upper layer of the model represents a wetlands, Kxx and Kyy represent overland flow resistance. Overland flow partially depends on the frictional resistance to flow, which in turn, depends on the topography of the wetlands base, depth of water, and nature of the vegetative cover. The "equivalent hydraulic conductivity" for a cell representing a wetlands is several orders of magnitude higher than the hydraulic conductivity of a cell representing even the most permeable aquifer. Similarly, Kzz represents vertical conductivity from the center (node) of an aquifer cell to the center of an aquifer cell below or above it. When the top layer of the model represents a wetlands, Kzz effectively represents the vertical conductivity from the bottom of the wetlands cell (representing land surface) to the center of the aquifer cell below it.

The last term in equation 1 that must be considered is the specific storage term S_S. In a water-table aquifer, specific storage is equal to the effective porosity of the aquifer divided by a unit change in head; however, in a wetlands, the equivalent porosity approaches 1, depending on the nature and extent of the vegetation. Specific storage is also of interest in another context when the top layer of the model represents a wetlands; that of resetting heads in cells that are rewetting after having gone dry in an earlier timestep.

All of the cells in the model are initialized as constant head, variable head, or no-flow cells by an input array referred to as the IBOUND array. When a variable head cell becomes dry, it is made a no-flow cell by resetting the IBOUND array element to 0 for that cell. The conductance coefficients representing flow to or from a no-flow cell are deleted for all cells adjacent to the no-flow cell. When a cell rewets, BCF2 resets the IBOUND array element to 1 (the code for a variable head cell).

Rewetting of cells is based on the five cells below and adjacent to the dry cell or only on the cell below. The cell is rewet when head in the neighboring cell(s) is higher than the bottom elevation of the dry cell plus some threshold value. Optionally, rewetting can be controlled to occur only at a selected iteration interval.

Once the decision criterion for rewetting is met, the heads in the rewet cell are calculated by one of two equations:

$$h = BOT + WETFCT(hn - BOT) \tag{2}$$

or

$$h = BOT + WETFCT(THRESH) \tag{3}$$

where,

BOT is the bottom elevation of the dry cell,

hn is the head in the neighboring cell causing rewetting,

$WETFCT$ is a factor supplied by the user to adjust assigned head value, and

$THRESH$ is the threshold value for rewetting.

When a wetlands is represented in the model, $WETFCT$ can be used to reflect the change in effective porosity between the wetlands layer (having an equivalent porosity of close to 1) and the aquifer layer. A more thorough discussion of BCF2 and the selection of alternate criteria for wetting cells and assigning heads is presented by McDonald and others (1991). The use of BCF2 and MODFLOW in representing a wetlands is discussed in a companion paper that is currently in preparation (Barbara Howie and E.D. Swain, U.S. Geological Survey, written commun., 1996).

The Evapotranspiration Package (EVT1 and EVT2)

The effects of evaporation and transpiration on ground water are accounted for in MODFLOW by the

EVT1 package (McDonald and Harbaugh, 1988). The algorithm functions in accordance with the following rules:

- When the ground-water head is at or above a certain level, termed the evapotranspiration surface, evapotranspiration loss occurs at a maximum rate, defined by the user.
- When the ground-water head drops below a certain extinction depth (defined by the user) from the evapotranspiration surface, evapotranspiration ceases.

Between these limits, the evapotranspiration rate varies linearly with ground-water head. The equation forms are:

$$R_{ET} = R_{ETM} \qquad h > h_s \tag{4}$$

$$R_{ET} = 0 \qquad h < h_s - d \tag{5}$$

$$R_{ET} = R_{ETM}\left(\frac{h - (h_s - d)}{d}\right) \qquad (h_s - d) \leq h \leq h_s \tag{6}$$

where,

R_{ET} is the rate of loss per unit surface area due to evapotranspiration,

R_{ETM} is the maximum rate of loss due to evapotranspiration,

h is the head in the aquifer,

h_s is the evapotranspiration surface elevation, and

d is the extinction depth.

The finite-difference formulation divides the aquifer into sections or "cells" arranged in layers, rows, and columns. The loss due to evapotranspiration must be taken from specific cells (usually the cells in the top layer). The EVT1 package allows two user-specified options: option 1 allows evapotranspiration to be drawn from the top layer only, and option 2 allows the user to define the layer from which evapotranspiration will be drawn for each vertical column. This definition is made for every stress period (time of constant boundary conditions).

If a cell from which evapotranspiration is drawn becomes dry, no evapotranspiration occurs. The assumption is that once the ground-water head drops below the bottom of the highest layer, the head is below the extinction depth and no evapotranspiration can occur. Thus, no option was created in EVT1 to allow the cells from which evapotranspiration is drawn to vary vertically during a stress period.

There is an interesting analogy between the representations of evapotranspiration and recharge in MODFLOW. Recharge in MODFLOW is accounted for by the RCH1 package (McDonald and Harbaugh, 1988). In RCH1, the user-specified recharge is distributed to cells according to one of three user-selected options. Option 1 applies recharge to the top layer only (analogous to option 1 in EVT1). Option 2 allows the user to define the layer to which recharge is applied (analogous to option 2 in EVT1). However, unlike EVT1, RCH1 has option 3 where recharge is applied to the highest wet cell. In this scenario, recharge from rainfall to a dry cell in the top layer is represented as passing through the cell (infiltrating through the unsaturated zone) and reaching the underlying wet cell. Due to the natures of recharge and evapotranspiration, option 3 was seen as necessary for recharge but illogical for evapotranspiration.

The scenario described so far has all layers of the MODFLOW model representing parts of the aquifer, the original intent of the model. However, when the topmost layer of the model is used to represent overland flow, a reexamination of the dynamics of evapotranspiration in the model must be made. When the top layer runs dry, the ground-water head is at the top of the second layer, defined as land surface (fig. 5). The extinction depth, d, occurs somewhere below layer 1. However, for this scheme to work properly, evapotranspiration must be taken from layer 1 when it is wet and from layer 2 when layer 1 is dry. Thus, when using the uppermost aquifer layer in the model to represent overland flow, an option 3 is necessary in the evapotranspiration package analogous to option 3 in RCH1.

The modified evapotranspiration package is referred to as EVT2. Largely, it is identical to EVT1; options 1 and 2 work the same. When the user specifies option 3, EVT2 starts at the uppermost layer and searches downward for the highest wet cell from which evapotranspiration is drawn. If the second layer is dry as well as the first, EVT2 would specify layer 3 as the layer from which evapotranspiration is drawn. If layer 3 is below the extinction depth, no evapotranspiration would occur.

The MODBRANCH Package

The MODBRANCH package was developed (Swain and Wexler, 1993) to represent complex surface-water and ground-water interactions by modifying the BRANCH one-dimensional surface-water model to act as a subroutine of the MODFLOW three-dimensional ground-water model. MODBRANCH accounts for the leakage between the two systems. The BRANCH model was originally created to represent unsteady nonuniform flow in an interconnected network of open channels (Schaffranek and others, 1981; Schaffranek, 1987). The coupling to MODFLOW allows the representation of such a complex surface-water regime in hydraulic connection with a ground-water system. Because the Dade County flow system involves a highly regulated canal network with dominant backwater effects and rapid ground-water responses to surface-water changes, the MODBRANCH package was considered the only viable scheme to correctly model the hydraulic situation.

The data requirements for the MODBRANCH package are substantially larger than for the River and Stream packages (McDonald and Harbaugh, 1988). Channel geometry, network configuration, flow resistance specifications, and initial and boundary conditions must be defined for the simulation. Most of these BRANCH model input requirements are defined in Schaffranek and others (1981). Recent extensions and enhancements are available (R.S. Regan, U.S. Geological Survey, written commun., 1996). The extra input requirements to define the surface-water/ground-water leakages are described in Swain and Wexler (1993).

BRANCH defines the network configuration in terms of segments, branches, and junctions (their interrelation is shown in fig. 6). The segment is the smallest division of the open-channel network and is bounded by defined cross sections. The channel geometry is defined at each cross section by an input table that defines cross-sectional area and top width for varying stages. The segment length and Manning's roughness coefficient for the segment must also be defined. A branch can be comprised of one or more segments end to end, and the end points of a branch are termed junctions. A mathematical transformation is employed in the model that combines the flow

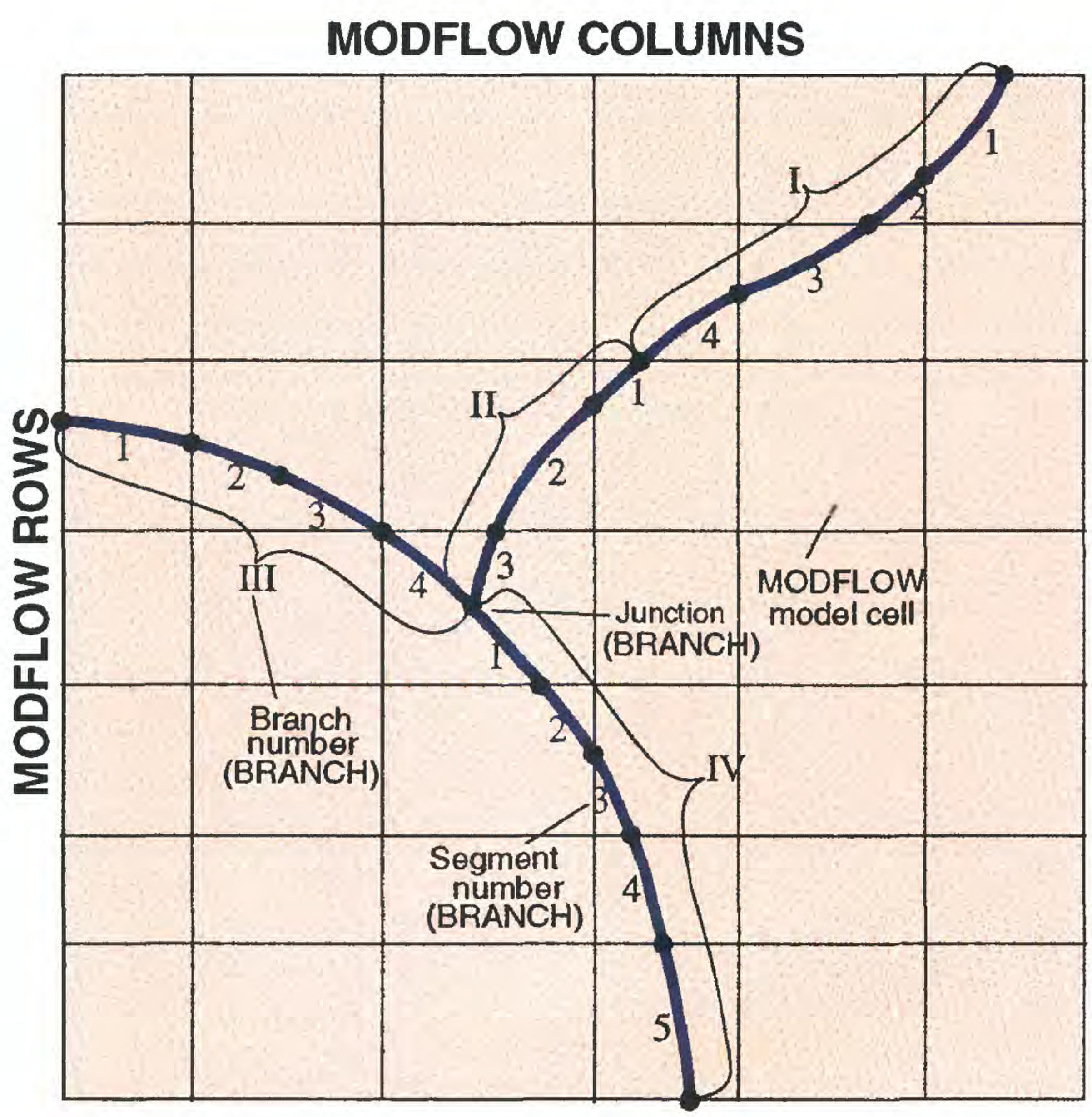

Figure 6. Relation of channel segments and aquifer blocks.

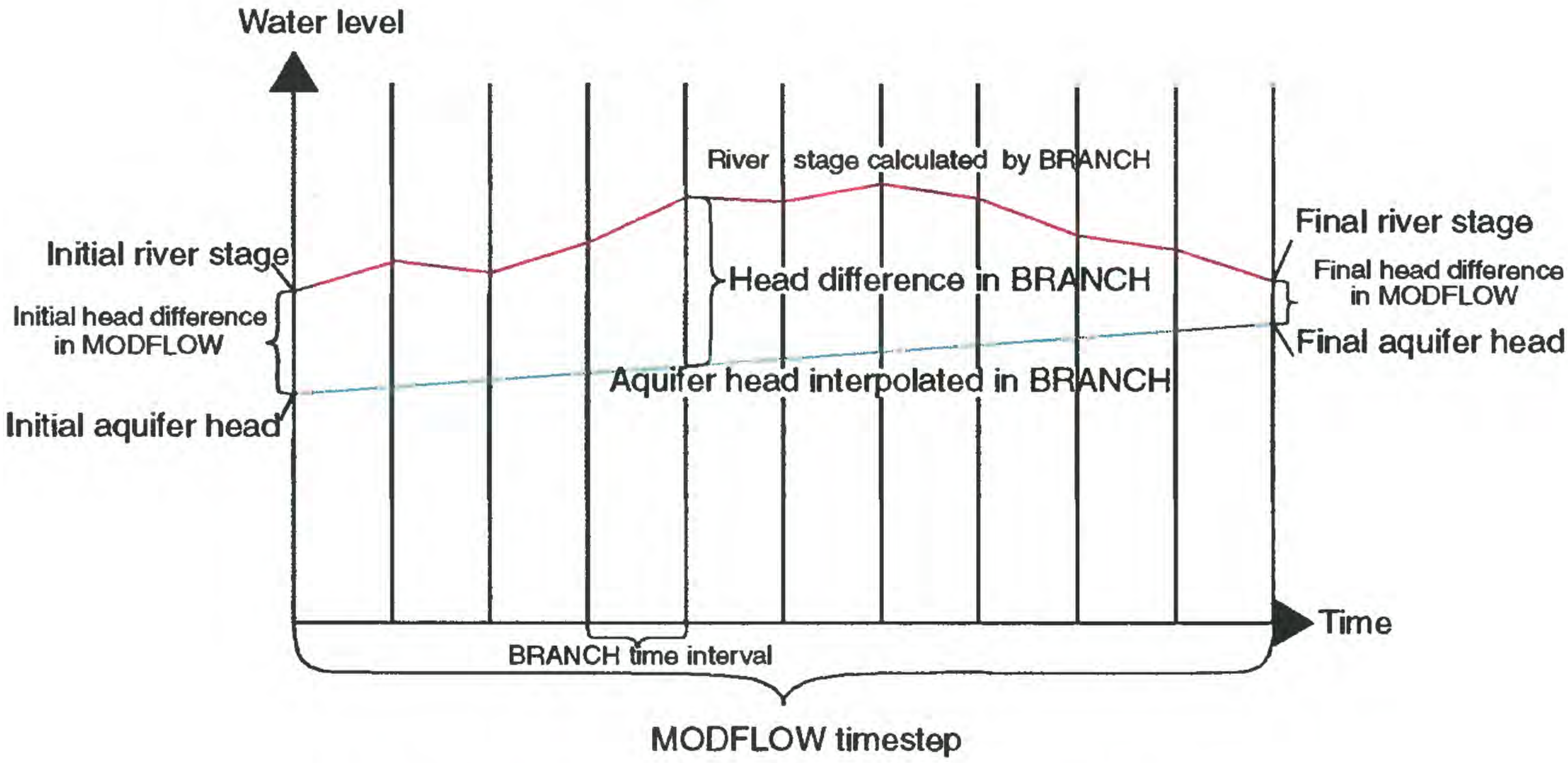

Figure 7. Relation of BRANCH and MODFLOW timesteps.

equations for all segments in a branch into one set of equations. This can greatly reduce the solution time in many circumstances. Because of this transformation, a branch cannot contain an internal inflow or outflow or a parallel loop. Multiple branches can connect at an internal junction, and point inflows and outflows can be specified at internal junctions.

In order for BRANCH to interchange leakage with MODFLOW through the MODBRANCH package, each channel segment must be assigned an aquifer block from which all leakage quantities occur. This requires that a BRANCH segment not span more than one MODFLOW model cell; therefore, a cross section must be defined wherever a surface-water channel crosses a ground-water model grid line (fig. 6). This necessitates defining the geometry of channel cross sections at points that usually do not correspond to surveyed locations. Thus, the ability to interpolate cross-sectional geometry is an important needed feature in the input data processing required for MODBRANCH.

Both the MODFLOW and BRANCH models solve their respective equations in finite-difference form, discretized in space and time. The timestep in each model can be different, with the BRANCH timestep typically shorter than the MODFLOW timestep. Because surface-water velocities are usually higher than ground-water velocities, shorter timesteps are generally used in BRANCH (fig. 7). The interrelation of the BRANCH and MODFLOW timesteps and water levels are shown in figure 7. Leakage quantities are calculated for each BRANCH timestep based on the water-level difference between the channel in BRANCH and the aquifer cell in MODFLOW and incorporated into MODFLOW so that the same leakage quantity is used in both models. The computation continues iteratively, within and between each model, until a common solution is reached.

The Streamlink Package

The Streamlink package was developed (Swain and Wexler, 1993) to represent hydraulic connectivity between the various packages in MODFLOW and MODBRANCH other than leakage (direct flows). Streamlink allows direct connections between BRANCH and the Stream and River packages or with an aquifer block in MODFLOW. The various connections are shown in figure 8. These connections allow the packages to work together, with various sections of an interconnected network of channels represented with BRANCH and the River and Stream packages. Streamlink also permits direct flow from a channel into a wetlands represented as the upper layer of the aquifer.

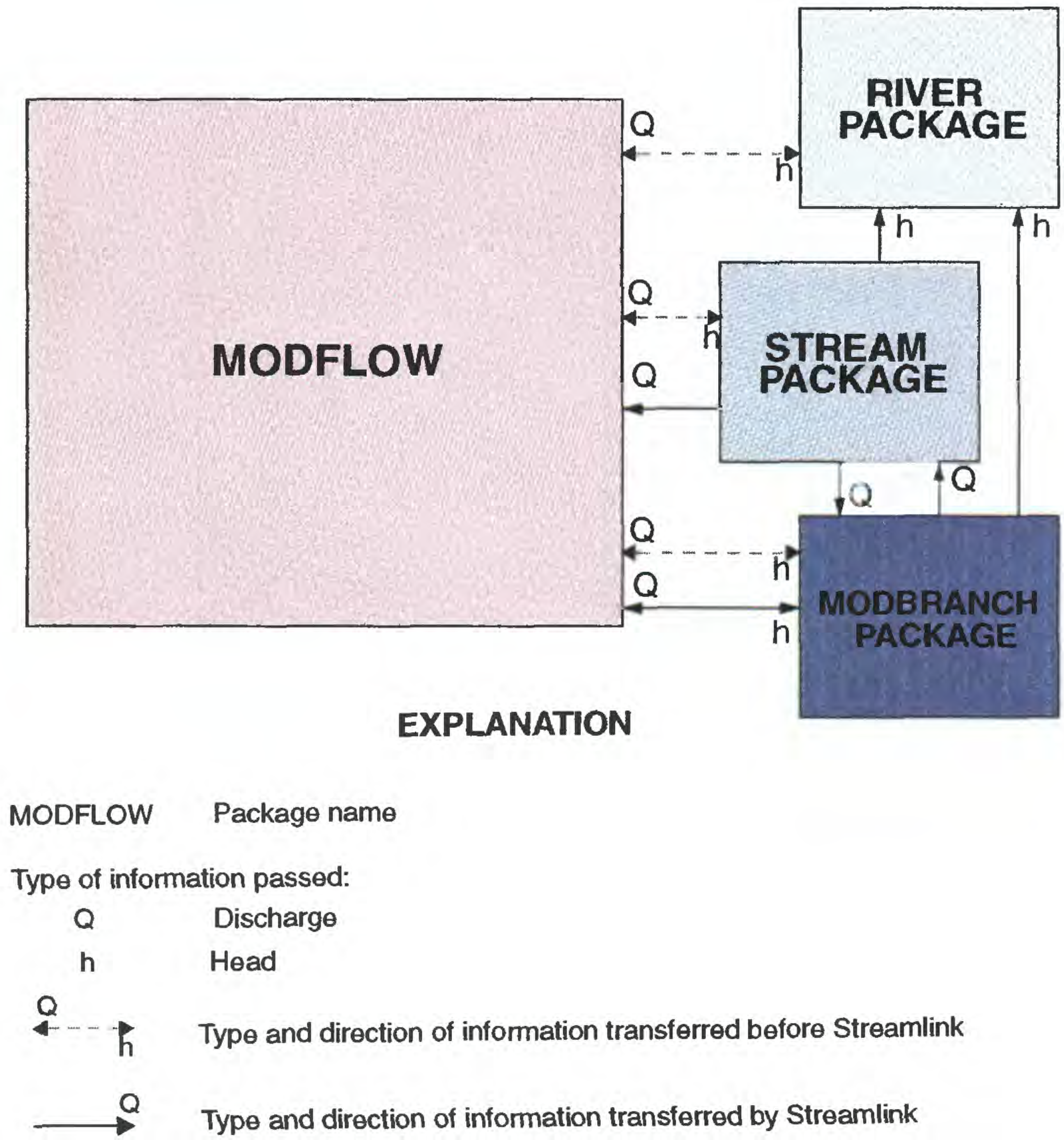

Figure 8. Connections between MODFLOW and the River, Stream, and MODBRANCH packages.

Incorporation of Hydraulic Structures into BRANCH

Because the surface-water system in Dade County is highly regulated by hydraulic structures, it is essential that the surface-water model has the capability to represent flow through these structures. The effects of hydraulic structures have been included in the solution schemes of open-channel flow models that are less complex than BRANCH (Fread, 1978; Hydrologic Engineering Center, 1982). The effects of tide gates have been incorporated into BRANCH (Goodwin, 1991) by allowing an internal junction to change between two boundary conditions: (1) the stage on either side of the junction being equal, and (2) no flow allowed through the junction. The first condition indicates an open gate with no head loss across the gate, and the second condition indicates a closed gate. This method does not allow for an intermediate gate opening, gates located other than at a junction, or structures other than a gate. Bower and others (1993) modified BRANCH to simulate flow through a full culvert with flap gates.

The simulation of gated spillways, gated culverts, and pump stations was required for the Dade County model. Consequently, a subroutine was created for the BRANCH model (Swain, 1992) to simulate flow through the variety of hydraulic structures encountered in Dade County. This subroutine sets appropriate coefficients in the solution matrix of

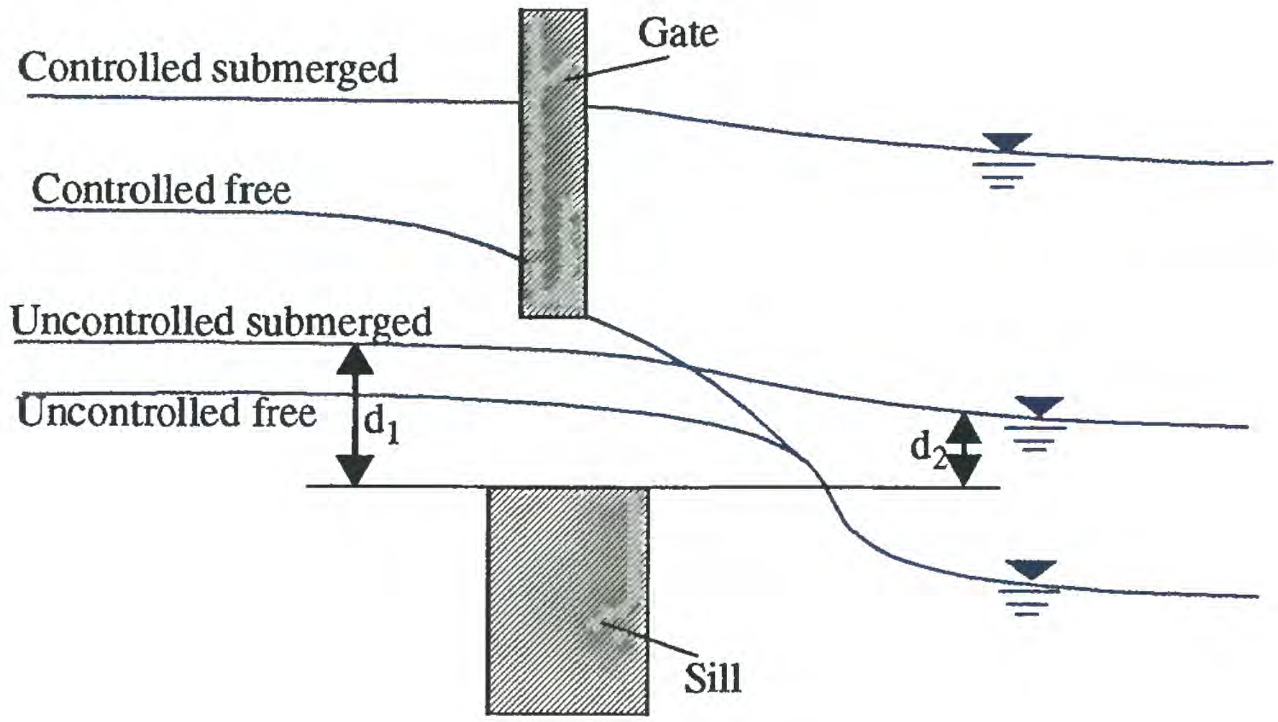

Figure 9. Flow regimes of gated spillway.

BRANCH to depict equations representing flow through various structures. The solution matrix has the same format (location of coefficients) as for channel flow alone, and no modification was required to the matrix solver. The continuity equation for an open-channel segment is formulated in the BRANCH solution matrix as (Schaffranek, 1987):

$$Q_{i+1}^{j+1} + \gamma Z_{i+1}^{j+1} - Q_i^{j+1} + \gamma Z_i^{j+1} = \delta \qquad (7)$$

where,

- Q is discharge,
- Z is stage,
- $i, i+1$ are cross-section locations,
- $j+1$ is the advanced time level, and
- γ,δ are coefficients derived from the finite-difference form of the continuity equation.

Nonzero values of γ and δ define the time rate of change of specific storage in the channel segment. These coefficients are set to 0 in the structure subroutine so that equation 7 represents flow through a structure with no internal specific storage.

The momentum equation for an open-channel segment is formulated in the BRANCH solution matrix as (Schaffranek, 1987):

$$\zeta Q_{i+1}^{j+1} + Z_{i+1}^{j+1} + \omega Q_i^{j+1} - Z_i^{j+1} = \varepsilon \qquad (8)$$

where ζ, ω, and ε are coefficients derived from the finite-difference form of the momentum equation.

A reformulation of equation 8, which defines the flow equation across a structure, is accomplished by setting ω and ε equal to 0, so that ζ is the flow coefficient relating stage difference to discharge through the structure. The previous discussion indicates that the composition of equations 7 and 8 for representing structures instead of channel segments simplifies the equations.

Gated Spillways

The representation of flow across a gated spillway in equation 8 requires the definition of flows both when the gate is in and out of the water. Collins (1977) defines flow at a gate in four regimes (fig. 9): free orifice, submerged orifice, free weir, and submerged weir. Due to the slow flows and small gradients, it is assumed that free orifice conditions do not exist at any of the structures in the model area. Free weir and submerged weirs can be represented by the same equation as opposed to adjusting the free weir equation for submergence by Collins (1977). This formulation starts with the energy equation, neglecting pressure head and friction losses:

$$d_1 + \frac{\alpha_1 \bar{v}_1^2}{2g} = d_2 + \frac{\alpha_2 \bar{v}_2^2}{2g} \qquad (9)$$

where,

- d_1 is upstream stage measured relative to the weir sill,

d_2 is downstream stage measured relative to the weir sill,

$\bar{v}_1$ is mean upstream velocity,

$\bar{v}_2$ is mean downstream velocity,

α_1 is upstream energy coefficient,

α_2 is downstream energy coefficient, and

g is gravitational acceleration.

After setting $\bar{v}_1 = Q/Ld_1$ and $\bar{v}_2 = Q/Ld_2$, where L is the length of the weir, and when rearranging, the following equation is obtained:

$$2gL^2(d_1 - d_2)d_1^2 = Q^2\left(\frac{d_1^2}{d_2^2}\alpha_2 - \alpha_1\right) \tag{10}$$

The energy coefficients can be expressed as (French, 1985):

$$\alpha = \frac{1}{d\bar{v}^3}\int_0^d v^3 dx \tag{11}$$

Redefining α_1 and α_2 in equation 10 by equation 11 yields:

$$2gL^2(d_1 - d_2)d_1^2 = Q^2\left(\frac{d_1^2}{d_2^2}\left[\frac{1}{d_2\bar{v}_2^3}\int_0^{d_2} v^3 dx\right] - \frac{1}{d_1\bar{v}_1^3}\int_0^{d_1} v^3 dx\right) \tag{12}$$

With $\bar{v}_1 = Q/Ld_1$ and $\bar{v}_2 = Q/Ld_2$, equation 12 becomes:

$$\begin{aligned} 2gL^2(d_1 - d_2)d_1^2 &= \frac{L^3}{Q}\left(d_1^2\int_0^{d_2} v^3 dx - d_1^2\int_0^{d_1} v^3 dx\right) \\ &= Q^2\left(\frac{L^3}{Q^3}d_1^2\int_{d_1}^{d_2} v^3 dx\right) \end{aligned} \tag{13}$$

Equation 13 reduces to:

$$Q = C_w L d_1\sqrt{2g(d_1 - d_2)} \tag{14}$$

where:

$$C_w = \sqrt{\left(\frac{Q^3}{A_1^2\int_{A_1}^{A_2} v^3 dx}\right)} \tag{15}$$

and where $A = Ld$ is the cross-sectional flow area. Normally, C_w is slightly less than 1.

Equation 14 is used to represent flow through a gated spillway when the gate is out of the water. It represents both submerged and free weirs in that when d_2 goes to 0, equation 14 reduces to the familiar free weir equation (Collins, 1977):

$$Q = C_w\sqrt{2g}Ld_1^{1.5} \tag{16}$$

When a gate is in the water, the submerged orifice equation is used (Collins, 1977):

$$Q = C_g L h_g\sqrt{2g(d_1 - d_2)} \tag{17}$$

where,

C_g is the submerged orifice coefficient for the gate, and

h_g is the gate opening.

Because L and g remain practically constant for a given gate and C_g is a function of gate opening, equation 17 can be expressed as:

$$Q = [C(h_g)]\sqrt{(d_1 - d_2)} \tag{18}$$

where the coefficient $C(h_g) = C_g L h_g\sqrt{2g}$ is only a function of gate opening h_g. $C(h_g)$ is calculated as a single number either from equation 18 with the values of Q, d_1, and d_2 from the previous timestep (if the gate is not moved), or calculated to pass the desired Q at the existing head difference d_1 - d_2 (if the gate is moved). Thus, an input value of C_g is not needed for the actual flow computations; it is assumed that a gate opening h_g is attained for the value of $C(h_g)$ to be as desired.

To check if a gate is out of the water, a computation is made to determine if the orifice equation (18) is allowing more discharge than would be possible with equation 14 for weir flow. If Q computed by equation 18 exceeds Q computed by equation 14, equation 14 is used and it is assumed that the gate is high enough to be out of the water or to allow for effective weir flow. This check allows smooth transitions between the two simulated flow regimes.

To determine if a gate is to be moved for a new timestep, a comparison of the upstream stage is made to user input criteria. This method is used because the structures in southern Florida mostly operate automatically based on upstream water levels. More water is released when the upstream level rises above a defined level, and less water is released when the upstream level drops below a defined level. The user-defined three stages are: minimum desirable, optimal, and maximum desirable (defined in operation rules). If the stage either exceeds the maximum desirable or drops below the minimum desirable, a new value of $C(h_g)$ is calculated so that d_1 is the desired value for the present values of Q and d_2. $C(h_g)$ is recalculated every iteration because the present values of Q, d_1, and d_2 change from iteration to iteration. If $C(h_g)$ drops to 0, it is assumed that the gate is closed. By comparing equations 14 and 18, it can be seen that if $C(h_g)$ exceeds $C_w L d_1 \sqrt{2g}$, weir flow is assumed and the gate is (effectively) out of the water.

To incorporate equations 14 and 18 into the BRANCH solution matrix equation (8), the coefficients are set as follows. For gate-in-the-water conditions (eq. 18), $\zeta = Q/[C(h_g)]^2$, $\omega = 0$, and $\varepsilon = 0$. It can be seen that because $d_1 - d_2 = Z_1 - Z_2$, equation 8 reduces to equation 18 with these values of the coefficients. For gate-out-of-the-water conditions (eq. 14), $\zeta = Q/2g(C_w L d_1)^2$, $\omega = 0$, and $\varepsilon = 0$.

Culverts

Fully submerged culverts and gated culverts can be represented in the structure subroutine. Culverts are generally rated by the equation (French, 1985):

$$Z_1 - Z_2 = C_c \frac{v^2}{2g} \qquad (19)$$

where C_c is the resistance coefficient for the culvert. The BRANCH matrix equation (8) expresses equation 19 when $\zeta = QC_c/2gA_c^2$, $\omega = 0$, and $\varepsilon = 0$, where A_c is the cross-sectional area of the culvert. A multiple barrel culvert can be represented, if each barrel has the same value of C_c, by multiplying the value of A_c by the number of barrels. If the culvert is gated:

$$Z_1 - Z_2 = C_c \frac{v^2}{2g} + \left(\frac{Q}{C_g A_g \sqrt{2g}} \right)^2$$
$$= Q^2 \left(\frac{C_c}{2gA_c^2} + \frac{1}{C_g^2 A_g^2 2g} \right) = Q^2 C_{cg} \qquad (20)$$

where,

A_g is the orifice area exposed by the gate, and

C_{cg} is the combined resistance coefficient for the gate and culvert.

As is similar to the gated spillway case, the coefficient C_{cg} can be calculated from the values of Z_1, Z_2, and Q in the previous timestep (if the gate is not moved) or from the desired value of Z_1 for the computed Z_2 and Q (if the gate is moved). Each time C_{cg} is computed, a verification is made to determine if it corresponds to a fully open gate condition. Because the orifice area exposed by the gate A_g cannot exceed the cross-sectional area of the culvert A_c, C_{cg} must be greater than $(C_c + 1/C_g^2)/(2gA_c^2)$. If it is not greater, the gate is assumed fully open and C_{cg} is set to $(C_c + 1/C_g^2)/(2gA_c^2)$. The procedure to determine if the gate is to be moved for a new timestep is the same as described for the gated spillway. In this gated culvert case, the coefficients in equation 8 are set to $\zeta = QC_{cg}$, $\omega = 0$, and $\varepsilon = 0$.

Pumps

The effects of a pump in the canal network can be represented by modifying the orifice flow equation (18) to account for an additional head energy added by the pump. The expression becomes:

$$Q = C_p \sqrt{Z_1 - Z_2 + h_p} \qquad (21)$$

where,

C_p is a coefficient of resistance for the pump when siphoning, and

h_p is the head added by the pump.

C_p is calculated from a user input design head difference, Δh_{design}, and design gravity flow, Q_{grav}, by $C_p = Q_{grav}/\sqrt{\Delta h_{design}}$. If the upstream stage exceeds a user-defined maximum, Z_{max}, pumping occurs and a nonzero value of h_p is calculated. To maintain the upstream stage, $h_p = (Q/C_p)^2 + Z_2 - Z_{max}$. This expression solved with equation 18 will keep Z_1 at Z_{max}. This computation is performed only if Z_1 exceeds Z_{max} in the previous timestep.

There are two options relating to pump capacity. In the first option, the pump can be limited to a maximum pumpage equivalent to a design flow, Q_{pump}, at a design head difference Δh_{design}. Under this option, if h_p is greater than $(Q_{pump}/C_p)^2 + \Delta h_{design}$, h_p is set to $(Q_{pump}/C_p)^2 + \Delta h_{design}$. Alternately, in the second option, the pump may be given unlimited capacity in which case h_p is always $(Q/C_p)^2 + Z_2 - Z_{max}$ if Z_1 exceeds Z_{max} in the previous timestep. This alternative is useful in pump design where the maximum necessary pumping rate indicates the required pump size. The pump equation is incorporated into the BRANCH matrix equation 8 by setting $\zeta = Q/C_p^2$, $\omega = 0$, and $\varepsilon = h_p$.

DESCRIPTION AND FIELD ANALYSIS OF THE COUPLED MODFLOW/BRANCH MODEL

As previously stated, the coupled MODFLOW/BRANCH model was developed to test techniques for more accurately representing wetlands and surface water in the MODFLOW ground-water flow model. The next two major sections of this report present a description of the coupled MODFLOW/BRANCH model and its application to a field problem. The first section describes the components and characteristics of the model (model discretization and the representation of boundaries, hydraulic properties, canals, rainfall, and evapotranspiration). The second section presents the results of applying the coupled MODFLOW/BRANCH model and provides a comparison with other model results. Stations used to establish boundary conditions for the MODFLOW and BRANCH models are presented in table 2, rainfall stations used to determine recharge are presented in table 3, and stations used to calibrate the MODFLOW and BRANCH models are presented in table 4.

Description of the Coupled MODFLOW/BRANCH Model

The modeled area is south of Tamiami Canal and east of the Dade-Monroe County line (fig. 1). The northeastern boundary extends to the easternmost salinity control structure (S-22), and the southeastern and southern boundaries approximately extend to the coastline (fig. 4). The modeled area includes ENP west of the L-31N levee system and tidal wetlands to the south and southeast. The surface-water bodies represented in this field application include wetlands, canals, and structures (pumps, salinity control gates, levees, and stage and flow-divide gates). The grid, layer definitions, and aquifer characteristics were modified from a calibrated SWIP model of the area developed by Merritt (1995).

Model Discretization

The lateral discretization of the model consists of 23 rows ranging in width from 1,350 to 14,850 ft and 27 columns ranging in width from 800 to 15,750 ft (fig. 10). A narrow 100-ft wide column was added to the model grid to represent constant head cells east of the modeled area because flows to the ocean from canals represented by BRANCH could not be added to the water budget if the canals discharged to constant head cells. The emphasis of the model was to examine the interaction between the canals and aquifer using the coupled ground-water/surface-water model and to test the modifications in BRANCH that control flow in the canals. For this reason, the grid cells are narrow (800-1,200 ft) in the vicinity of long reaches of major canals and levees, such as L-67 Extension, L-31N, and C-111. Leakage into a model cell from a canal is greatly affected by canal geometry and canal stage. Therefore, a grid line was located in places where canal geometry or acute stage changes occurred, such as at structures or canal junctions. For example, grid lines were added to the original grid of Merritt (1995) for structures S-331 and S-332 and for the junctions of Black Creek Canal (C-1) and L-31N Canal and of C103S and the main channel of the Mowry Canal (C-103). In areas distant from canals and in the areas of lowest ground-water head gradient, model cells are several miles on a side.

The vertical discretization of the modeled area is shown in figure 11. The model consists of one layer representing a wetlands and two layers representing the Biscayne aquifer. Layer 1, the wetlands layer, extends from land surface (0-11 ft above sea level) to 15 ft above sea level. The maximum 15 ft was arbitrarily selected because it is higher than the maximum land-surface elevation plus maximum depth of water in the wetlands. Those cells in layer 1 that represent the higher areas of the Atlantic Coastal Ridge are set inactive in MODFLOW's IBOUND array because these areas are never submerged (fig. 10). A string of cells in column 10 and the cells representing the Tamiami Canal (row 2) are also inactive because of a levee and a road that impede surface-water flow, respectively.

Layer 2 extends from land surface to the base of the Miami Limestone and includes the low-permeability peats and marls present in the northwestern and southern parts of the study area. The thickness of layer 2 ranges from 2 ft in the southernmost part of the study area to 30 ft below the Atlantic Coastal Ridge in the northeastern part. The top and bottom elevations of layer 2 were obtained from the model developed by Merritt (1995) in which land surface was determined from available land-surface maps and the base of the Miami Limestone was assigned from a reconnaissance study by Fish and Stewart (1991).

Table 2. Stations used to establish boundary conditions for the ground-water flow model (MODFLOW) and the surface-water flow model (BRANCH)

[Type of station: GW, ground water, SW, surface water, TD, tidal. Source of data: ENP, Everglades National Park; USGS, U.S. Geological Survey; SFWMD, South Florida Water Management District. Other abbreviations: (D), downstream; (U) upstream. The identification number for USGS stations is used to retrieve data from the USGS NWIS data bases. The identification number for ENP and SFWMD stations is the latitude-longitude]

MODFLOW				BRANCH			
Station number or name	Type of station	Identification number	Source of data	Station number	Data type	Identification number	Source of data
EP-GW	GW	251646 0803024	ENP	S-334 (D)	Stage	254538 0803003	SFWMD
F-179	GW	254444080144801	USGS	S-197 (U)	Stage	251712 0802630	SFWMD
F-319	GW	254217080171801	USGS	C-2 Canal at Tamiami Canal	Stage	02289500	USGS
G-580	GW	254000080181001	USGS	S-123 (D)	Stage	253636 0801829	SFWMD
G-860	GW	253718080192301	USGS	S-21 (D)	Stage	253234 0801952	SFWMD
Craighead Lake	SW	251344 0804215	ENP	S-21A (D)	Stage	253108 0802047	SFWMD
NP-205	SW	254120 0805057	ENP	S-20F (D)	Stage	252745 0802052	SFWMD
P-33	SW	253652 0804211	ENP	S-22	Discharge	254011 0801703	SFWMD
P-34	SW	253630 0805628	ENP	S-332	Discharge	252518 0803524	SFWMD
P-35	SW	252739 0805156	ENP				
P-37	SW	251708 0804119	ENP				
P-38	SW	252212 0805000	ENP				
S-20 (D)	SW	252200 0802236	SFWMD				
S-20F (D)	SW	252745 0802052	SFWMD				
S-21 (D)	SW	253234 0801952	SFWMD				
S-123 (D)	SW	253636 0801829	SFWMD				
S-197 (D)	SW	251712 0802630	SFWMD				
Tamiami Canal	SW						
L-30 to L-67A	SW	02289060	USGS				
Near Coral Gables	SW	02289500	USGS				
S-12A (D)	SW	254543080491101	USGS				
S-12B (D)	SW	02289019	USGS				
S-12C (D)	SW	02289041	USGS				
S-12D (D)	SW	254543080405400	USGS				
S-334 (U)	SW	254538 0803003	SFWMD				
S-334 (D)	SW	254538 0803003	SFWMD				
Florida Bay at Flamingo (long-term average)	TD	02290825	USGS				

Table 3. Rainfall stations with sufficient continuous data to use in determining recharge

[SFWMD, South Florida Water Management District; ENP, Everglades National Park; NOAA, National Oceanic and Atmospheric Administration]

Station number or name	Station used to determine recharge to the model	Latitude-longitude number	Source of Data
Chekika	Yes	253656 0803503	SFWMD
EPR	No	251609 0803017	ENP
Miami International	Yes	254900 0801700	NOAA
NESRS1	Yes	254130 0803805	ENP
NP-201	Yes	254305 0804333	ENP
NP-203	Yes	253725 0804422	ENP
NP-206	Yes	253242 0804022	ENP
P-35	Yes	252739 0805156	ENP
P-36	Yes	253139 0804745	ENP
P-38	Yes	252212 0805000	ENP
ROYALPLM	No	252310 0803539	SFWMD
R-3110	Yes	252547 0803835	ENP
S-18C	Yes	251914 0803131	SFWMD
S-20F	Yes	252745 0802052	SFWMD
S-331	Yes	253636 0803038	SFWMD
S-332	No	252518 0803524	SFWMD
S-336	Yes	254539 0802925	SFWMD

Layer 3 extends from the base of layer 2 to the base of the Biscayne aquifer, except in the northwestern part of the study area where the Biscayne aquifer does not exist. Layer 3 includes the highly permeable sediments of the Fort Thompson Formation throughout most of the study area; however, less-permeable sediments of the Fort Thompson Formation are included to the northwestern part as are sediments of the Anastasia Formation in the northeastern part and the Key Largo Limestone in the far southeastern part. The thickness of layer 3 ranges from 5 ft in the northeastern part of the study area to 90 ft in the eastern part (fig. 11). The base of layer 3 was obtained from the model of Merritt (1995) and is also based on the work of Fish and Stewart (1991).

Temporal discretization is treated separately by the MODFLOW and BRANCH models. The final ground-water or MODFLOW model used stress periods of 15 days and timesteps of 5 days. Initially, stress periods of 30 days were used to correspond to the monthly summaries available for most model input (rainfall recharge, well pumpage, canal stage, and ground-water head). This 30-day stress period worked well for all stress data, except rainfall recharge. Rainfall is highly variable, and because the unsaturated zone and aquifer are very porous, the effects of rainfall on the water table are rapid and significant. For this reason, using average daily rate for each month for rainfall recharge caused calibration problems, and 15 average daily rates and corresponding 15-day stress periods were used. In the BRANCH part of the model, 12-hour timesteps were used.

All data, other than rainfall, used by MODFLOW were based on the average daily values for each month, calculated by dividing monthly totals by days in the month. These include data for the River, Recharge, Well, Evapotranspiration, and General-Head Boundary packages. Flow and stage data for BRANCH are based on daily averages.

Table 4. Stations used to calibrate the ground-water flow model (MODFLOW) and the surface-water flow model (BRANCH)

[Type of station: GW, ground water, SW, surface water. Source of data: ENP, Everglades National Park; USGS, U.S. Geological Survey; SFWMD, South Florida Water Management District. Other abbreviations: (U) upstream. The identification number for USGS stations is used to retrieve data from the USGS NWIS data bases. The identification number for ENP and SFWMD stations is the latitude-longitude]

MODFLOW				BRANCH			
Station number	Type of station	Identification number	Source of data	Station number	Data type	Identification number	Source of data
EP-GW	GW	251646 0803024	ENP	S-20F	Discharge	252745 0802052	SFWMD
F-358	GW	252829080285101	USGS	S-21 (U)	Stage/Discharge	02290710	USGS
G-553	GW	253902080202501	USGS	S-21A (U)	Stage/Discharge	253108 0802047	SFWMD
G-596	GW	253937080304001	USGS	S-118 (U)	Stage	253721 0802031	SFWMD
G-614	GW	253258080264301	USGS	S-120 (U)	Stage	254014 0801918	SFWMD
G-620	GW	254000080460001	USGS	S-122 (U)	Stage/Discharge	253538 0802054	SFWMD
G-757A	GW	253537080284401	USGS	S-123 (U)	Stage/Discharge	253636 0801829	SFWMD
G-855	GW	254038080280201	USGS	S-148 (U)	Stage	253411 0802259	SFWMD
G-858	GW	253854080242801	USGS	S-165 (U)	Stage	253232 0802435	SFWMD
G-1183	GW	252918080234201	USGS	S-174	Discharge	252900 0803349	SFWMD
G-1251	GW	251922080340701	USGS	S-175 (U)	Stage/Discharge	252501 0803425	SFWMD
G-1502	GW	252656080350301	USGS	S-176 (U)	Stage	252857 0803347	SFWMD
G-3356	GW	252502080253901	USGS	S-177 (U)	Stage/Discharge	252407 0803331	SFWMD
S-182A	GW	253549080214101	USGS	S-194 (U)	Stage/Discharge	253458 0802843	SFWMD
EVER1	SW	251946080254800	USGS	S-331 (U)	Stage/Discharge	253636 0803038	SFWMD
NESRS1	SW	254130080380500	USGS	S-338 (U)	Stage/Discharge	253937 0802850	SFWMD
NESRS2	SW	254315080331500	USGS				
NP-44	SW	252604 0804314	ENP				
NP-46	SW	251910 0804746	ENP				
NP-62	SW	252622 0804722	ENP				
NP-67	SW	251950 0803902	ENP				
NP-72	SW	252345 0804212	ENP				
NP-203	SW	253725 0804422	ENP				
NP-206	SW	253242 0804022	ENP				
P-33	SW	253652 0804211	ENP				
P-36	SW	253139 0804745	ENP				
R-3110	SW	252547 0803835	ENP				
TSB	SW	252405 0803625	ENP				

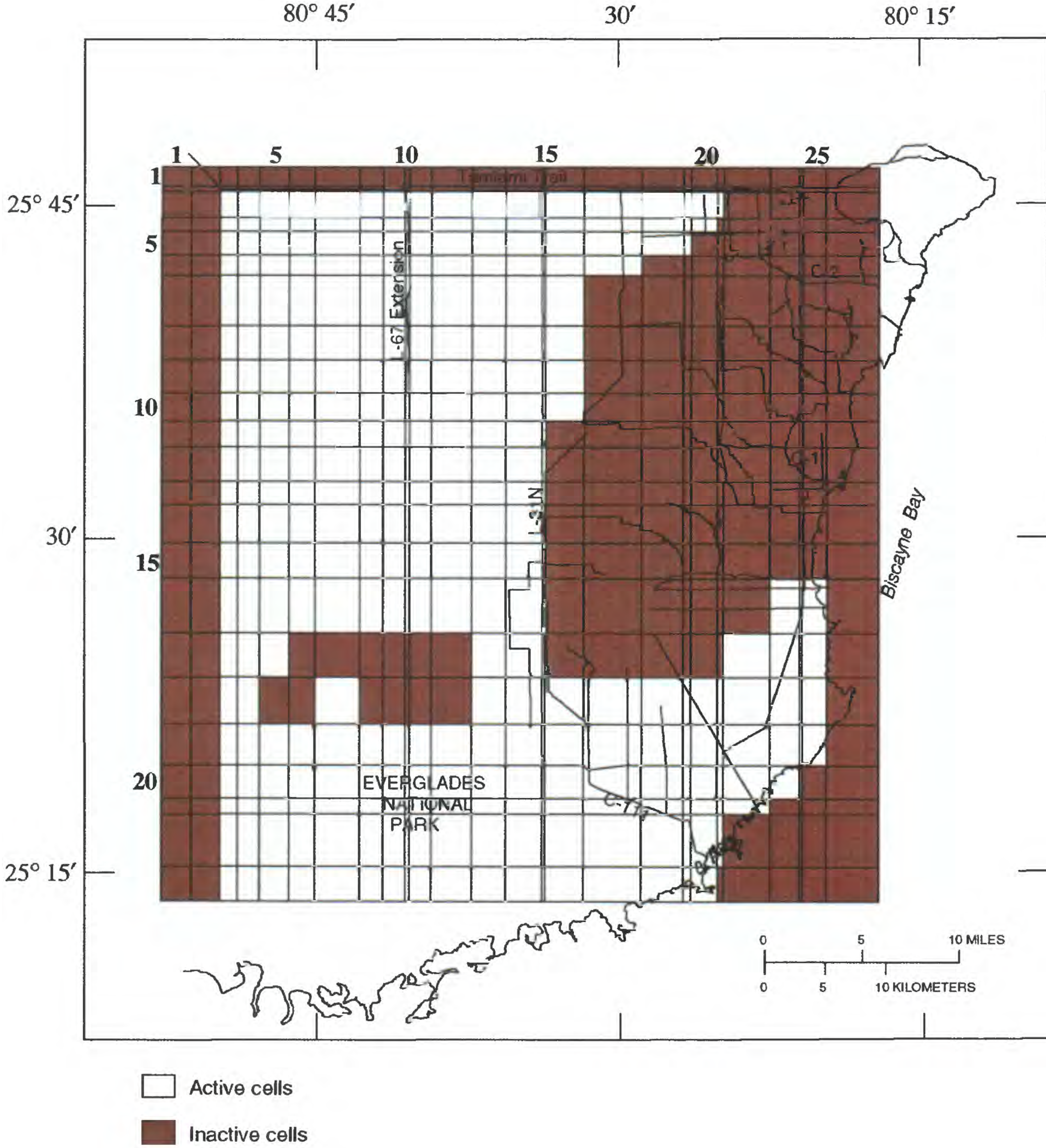

Figure 10. Model grid and inactive cells in layer 1.

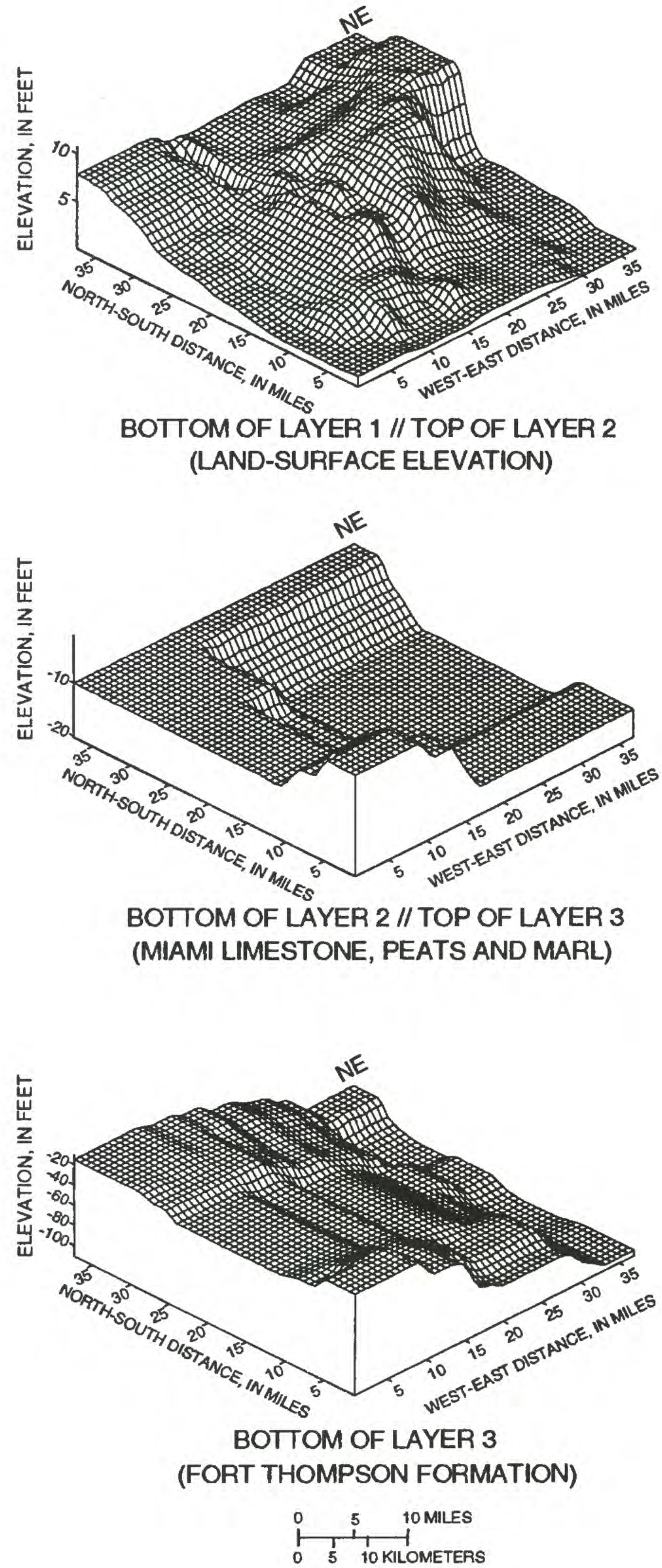

Figure 11. Layer elevations in model area.

Boundaries

The model boundaries, the ground-water and surface-water stations, and one tidal station used to establish boundary conditions are shown in figure 12. The lower boundary of the model is the less permeable unit of the Tamiami Formation. Several assumptions were made in assigning boundaries to the model. First, ground water and surface water are assumed to be in direct hydraulic connection. This assumption is supported by Merritt (1995) in an analysis of water levels at ground-water and surface-water stations in close proximity to each other and also is consistent with the observations of the authors. The second assumption is that the Tamiami Canal and associated levees act as surface-water and ground-water level controls. This assumption is supported by an analysis of long-term data from surface-water stations in the canal and nearby ground-water stations (Merritt, 1995). Lastly, at the time the model was developed, there was no package to simulate the saltwater interface in MODFLOW, so the interface is assumed to be vertical and to occur at the coastal boundaries. Heads assigned to this boundary are based on laterally interpolated values from coastal control structures (fig. 12).

Boundary assignments for layers 1, 2, and 3 are shown in figure 13. Layers 2 and 3 are identical. Row 1 and columns 1, 2, and 27 were inactive in the final calibrated model (representing no-flow boundary conditions). In layer 1 (the wetlands layer), the row that represents the Tamiami Canal, L-29, and the L-29 Canal (row 2) is inactive because the levees and higher elevation of the Tamiami Trail roadbed prevent any surface flow to the south from the water-conservation areas, except for minor flows through small culverts during the wet season. As previously mentioned, the cells that represent the Atlantic Coastal Ridge are also inactive in layer 1 because this area is never inundated. In layers 2 and 3, general head boundaries are assigned to cells in row 2 based on interpolated heads from surface-water stations in the Tamiami Canal and on well F-179 (fig 12).

Model boundaries to the west, south, and east are represented by general head boundaries in all layers (fig. 13). The values assigned to the western boundary are interpolated from data for S-12A downstream, wetlands stations, and the tidal station at Flamingo (fig. 12). The wetlands stations are constructed to continuously measure the water level as it moves cyclically from above to below land surface and back again as the wetlands intermittently dry and rewet. The values assigned to the southern boundary cells are interpolated from data for the tidal station at Flamingo, wetlands, surface-water, and ground-water stations. The eastern boundary is based on interpolated data from the Tamiami Canal at Coral Gables and ground-water wells to the north and on downstream stages at surface-water control structures to the south. A comparison of upstream and downstream stages to ground-water heads near this boundary indicated that the downstream stages most accurately estimated the head in the aquifer at the model boundary.

The BRANCH data set for the canals in the Dade County study area (fig. 4) contains 54 branches and 179 segments. Boundaries are defined as either specified stage, specified discharge, or no flow. Specified stage boundaries are located at the northern end of L-31N Canal, C-111 Canal upstream of structure S-197, Snapper Creek Canal (C-2) at Tamiami Canal (C-4), Cutler Drain Basin (C-100) downstream of S-123, Black Creek Canal (C-1) downstream of S-21, Princeton Canal (C-102) downstream of S-21A, and Mowry Canal (C-103) downstream of S-20F (fig. 4). Specified discharge boundaries are located at structures S-332 and S-22 (fig. 4). No-flow boundaries are located at the northern and southern ends of L-67 Extension Canal, southern end of L-31W Canal, northern end of C-111E Canal, northern end of C-100A, western end of C-100, northern end of L-31E, northern end of C-102N, L-31E at the ocean (blocked by a berm), northern end of C-103N, eastern end of C-113, and L-31E at S-20A (fig. 4).

Data for the stations used to assigned boundary conditions were obtained from the data bases of the USGS, the SFWMD, and ENP. Tidal stages at Flamingo are based on long-term averages calculated by Merritt (1995).

Hydraulic Properties

Lateral hydraulic conductivities assigned to model layers (fig. 14) are modified from values assigned in the calibrated model of Merritt (1995). Vertical hydraulic conductivities are set at one-tenth the lateral hydraulic conductivity. Merritt (1995) assigned most of layer 1, the wetlands layer, an "equivalent hydraulic conductivity" of 3,000,000 ft/d, which he determined from model calibration and supported by calculations using a modified version of the Manning equation developed by Chow (1964). In the

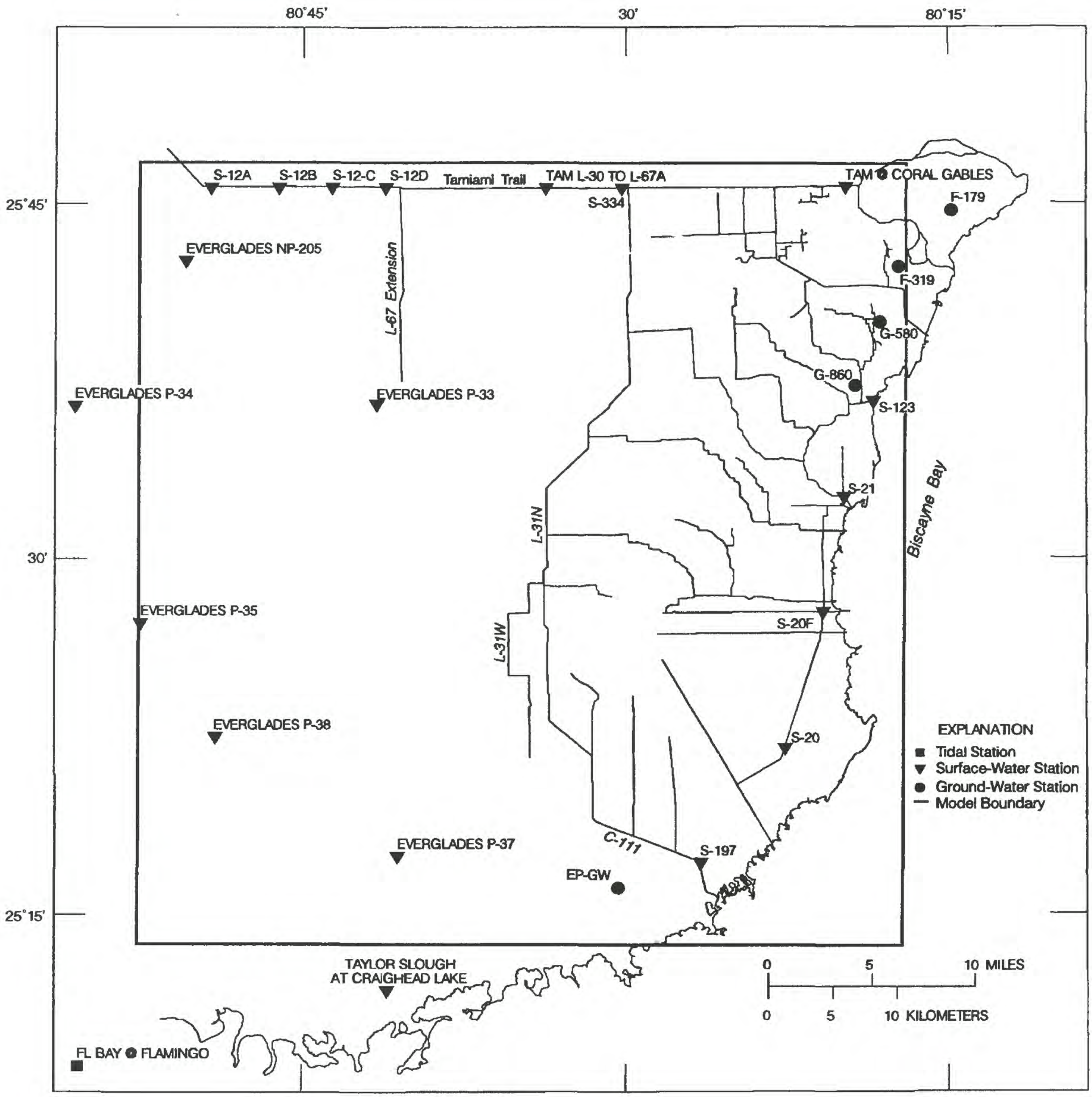

Figure 12. Stations used to establish boundary conditions.

Figure 13. Boundary assignments for the model layers.

southwestern part of the model area, Taylor Slough is deeper and offers less frictional resistance to flow. Therefore, model cells representing Taylor Slough are assigned a lateral hydraulic conductivity of 100,000,000 ft/d.

Layer 2 represents the aquifer from land surface to the base of the Miami Limestone and includes the peats and marls at the surface in much of the study area. Model cells representing areas where highly permeable Miami Limestone dominates the vertical column are assigned a lateral hydraulic conductivity of 30,000 ft/d. This value is based on aquifer-test analyses by Fish and Stewart (1991) and on the calibrated model of Merritt (1995). Model cells representing the northwestern part of the study area, where low permeability peats and marls dominate the relatively thin vertical column, are assigned a lateral hydraulic conductivity of 10 ft/d based solely on the calibrated model of Merritt (1995) because no test data are available for these deposits. In the north-central and southwestern parts of the study area, the peats and marls are present but are thin relative to the Miami Limestone. Model cells representing these areas are assigned a lateral hydraulic conductivity of 20,000 ft/d. Model cells which include the Anastasia Formation in the extreme northeastern part of the modeled area are also assigned a lateral hydraulic conductivity of 20,000 ft/d based on work by Fish and Stewart (1991, p. 33). Fish and Stewart (1991, p. 40) also determined an area in the vicinity of Snapper Creek and Black Creek Canals in the east where the Miami Limestone is less permeable than in other areas, possibly due to the filling of cavities with sand and silt. This area was assigned a lateral hydraulic conductivity of 5,000 ft/d by Fish and Stewart (1991), which is the value used in the model cells that represent that area.

The lateral hydraulic conductivity assignments for layer 2 that have been discussed, thus far, were based on the hydrogeology of the Miami Limestone, peats, and marls. In the agricultural area in southeastern Dade County, the previously mentioned network of shallow drainage canals required an upward adjustment of lateral hydraulic conductivities. Merritt (1995) used this method to represent the area of drainage canals, assigning a lateral hydraulic conductivity of 1,000,000 ft/d to the aquifer in this area. Because the layers in this model do not correspond exactly to those in Merritt (1995), this lateral hydraulic conductivity value was determined to be too high and was modified during the calibration process. The lateral hydraulic conductivity finally assigned to the area of drainage canals was 100,000 ft/d.

Layer 3, which extends to the base of the Biscayne aquifer, is comprised mainly of the Fort Thompson Formation with subjacent deposits of the Tamiami Formation. Most of layer 3 is highly permeable and is assigned a lateral hydraulic conductivity of 30,000 ft/d, reflecting the results of Fish and Stewart (1991). The area to the northwest, which Fish and Stewart (1991) determined to be less permeable, was assigned a lateral hydraulic conductivity of 500 ft/d based on the calibrated model of Merritt (1995). As in layer 2, the area of relatively low permeability in the vicinity of Black Creek Canal (C-1) and Snapper Creek Canal (C-2) determined by Fish and Stewart (1991) was assigned a lateral hydraulic conductivity of 5,000 ft/d.

Canal Representation

Several digital spatial data (DSD) layers were created to provide data for a model of the southern Dade County area and include the line DSD layer of canals, point DSD layer of canals, polygon DSD layer of the model grid, and the point DSD layer of control structures. The MODFLOW program used a line DSD layer to represent the location and geometry of the canals, and the BRANCH program used a point DSD layer to represent the location and geometry of the canals (both programs require a different formation DSD layer).

The Modelgrid program, written in Arc Marco Language (AML) by Daniel Winkless (U.S. Geological Survey, written commun., 1988), was used to create the grid DSD layer for southern Dade County. The program created three DSD layers of polygons, lines, and points. The layer, row, column area, and perimeter were attributes from the polygon DSD layer.

1. <u>River package</u>

   Digital Line Graph (DLG) data maps produced by the USGS National Mapping Division from 1:100,000 topographic maps were converted into DSD layers using ARC/INFO. The hydrography category was extracted from the DLG data to create the initial canal DSD layer. This DSD layer was updated by comparison to and digitizing from 1:24,000 scale topographic maps and later verified with aerial photographs taken in April 1990 by the Metro-Dade Department of Environmental Resources Management (DERM).

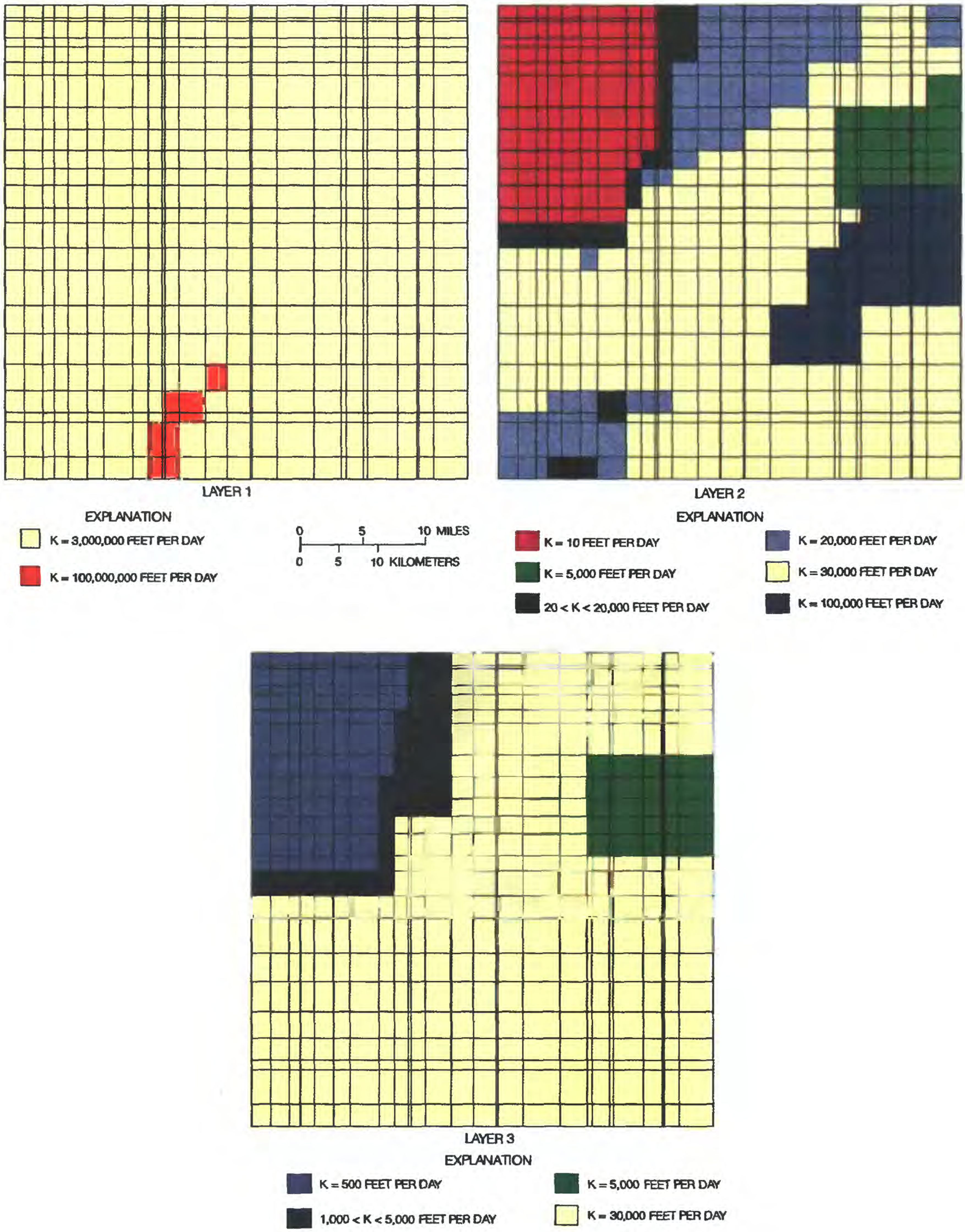

Figure 14. Lateral hydraulic conductivity assigned to each layer.

The cross-sectional data for primary canals included bottom elevation, bottom width, and canal side slope. These data were obtained from AS-BUILT, construction drawings, surveys, and design memoranda provided by the COE, SFWMD, and DERM (SFWMD personnel assisted in the research of the data). Cross-section surveys were conducted by USGS personnel to acquire data on Snapper Creek Canal (C-2). Data sources for secondary canals, obtained from DERM, included survey cross sections conducted every 10 years by DERM and AS-BUILT drawings of cross sections of canals.

The cross-sectional data were assigned to reaches of the canals based on differences in the canal geometry or stage. If changes in the geometry or stage were significant, then the canal was split and separate data values were assigned to each canal reach. Calculations were made with ARC/INFO to determine canal side length, wetted perimeter, and conductance. This DSD layer was intersected with the model grid to obtain the row and column of each canal reach. A computer program written in AML was used to generate data files needed by the River package of MODFLOW.

2. MODBRANCH package

The canal line DSD layer was used to create a point DSD layer of the canals for input into the MODBRANCH package. Also, data points were needed at junction endpoints, section endpoints, control structures, and points separating branches from canals that were represented with the River package (fig. 2). Multiple points exist at junctions and structure locations.

Each point was assigned a design stage, bottom elevation, bottom width, and side slope and related to the layer, row, and column in the MODFLOW model grid. In addition to these attributes, every point was assigned the value of the canal length from that point to the next downstream point. The last point in the branch was assigned a length of 0. Calculations were made to determine the area and top width of the canal at each cross-section point. Every point was designated by canal name, branch name, and branch number. Branches consist of single or multiple segments and two junctions, one junction at the beginning of the branch and one junction at the end (fig. 6), so as to distinguish an upjunction point and a downjunction point in each branch. Every intermediate point within a branch segment is assigned the sequence number of the upjunction and downjunction point for its branch. Also, every cross-section point was numbered. A computer program written in AML was used to generate data files needed by the MODBRANCH package.

Rainfall and Evapotranspiration

Data from rainfall stations maintained by the National Weather Service, the SFWMD, ENP, and the USGS were analyzed to compute recharge due to rainfall. Rainfall gaging stations were deleted from consideration if extensive data were missing or if data duplicated nearby stations. (Different agencies occasionally maintain stations at the same location because of wetlands accessibility problems.) Missing values were then estimated using the closest available gaging stations and, in some cases, a weighting or correction factor. The final set of 17 rainfall stations used during the modeling effort is shown in figure 15.

Rainfall in southern Florida is highly variable in both spatial distribution and intensity, especially during the summer wet season. Because of this variability, because each rainfall gaging station has some estimated period of record from other stations, and because of the scarcity of stations in the wetlands, the area represented by each station in the model was determined during model calibration rather than by geostatistical analysis. The rainfall gaging stations used in the calibrated model and the area represented by each station are shown in figure 16. In the shaded area in the northwestern and north-central parts of the study area, a factor of 0.4 was applied to rainfall recharge from June through August to account for losses by evaporation from the wetlands.

Evapotranspiration parameters (evapotranspiration surface, extinction depth, and maximum rates for each month) are based on calibrated values of the model of Merritt (1995). The evapotranspiration surface above which evapotranspiration occurs at the maximum rate was assigned as 0.1 ft in the peat and marl soils and 3 ft in the areas where limestone occurs at the surface. The assigned extinction depth below which no evapotranspiration occurs was 5 ft in the peat and marl soils and 20 ft in the areas where limestone occurs at the surface. The deeper-than-expected

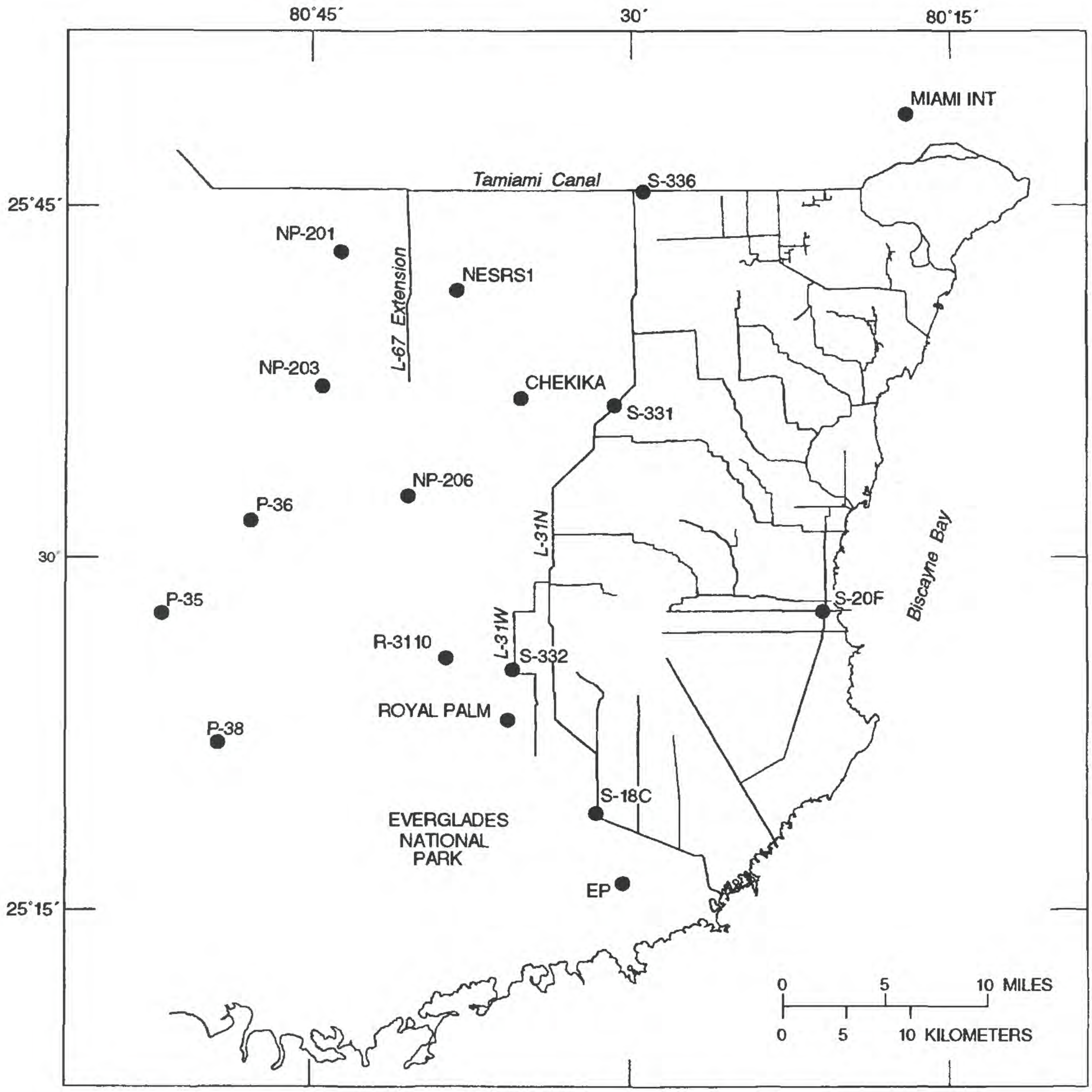

Figure 15. Rainfall stations with sufficient continuous record to use in the model.

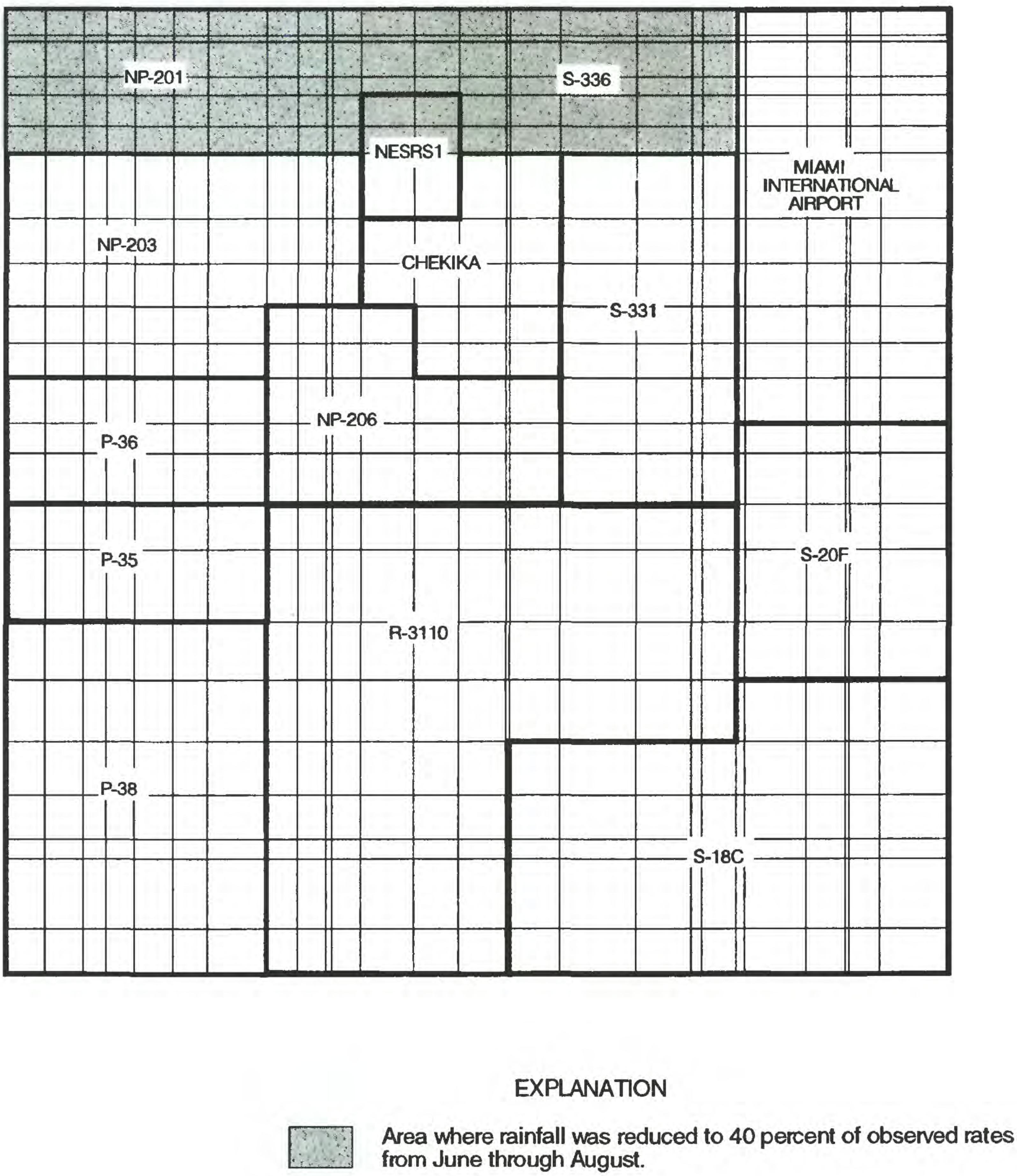

Figure 16. Model areas covered by each rainfall station.

extinction depths are supported by an analysis of recession rates in long-term hydrographs by Merritt (1995) in which no relation was found between evapotranspiration rates and the depth below land surface.

Field Analysis of the Coupled MODFLOW/ BRANCH Model and Other Model Results

The next sections of this report present the results of a field analysis of the coupled MODFLOW/ BRANCH model and a comparison with other model results. Calibration results and a sensitivity analysis are presented. A canal representation is made comparing the results of the MODBRANCH and River packages. Lastly, a wetlands representation is made comparing the results of the SWIFT2D and MODFLOW models.

Calibration Results

Calibration for 1990 conditions was achieved by varying those parameters which are known with the least certainty. The greatest effect on model results was achieved by varying evapotranspiration and the hydraulic structure operating rules, although sensitivity of Manning's n in the canals, leakage coefficients, and aquifer transmissivities were also evaluated. Evapotranspiration and rainfall rates tend to have counteracting effects; therefore, because rainfall rates are better known than evapotranspiration rates, evapotranspiration was selected as the calibration parameter. Evapotranspiration severely affects water levels in the wetlands, especially in those areas in the ground-water flow model (MODFLOW) distant from the damping effects of canals. The hydraulic structure operating rules are very important in meeting proper canal stages and discharges. As previously mentioned, the actual operation of a structure does not always match the written rules; therefore, the rules in the model were changed to match the actual structure operation for the calibration period.

Calibration involved varying the identifying parameters until observed and simulated water levels and discharges agreed reasonably well. An objective optimization-type calibration is beyond the scope of this study, but a sensitivity analysis (described in the next section) includes much of the statistical information that would be used in an optimization calibration. The results of the calibration run are shown in figures 17 to 20. The observed stages at coastal or upstream canal structure sites and simulated canal stages produced by BRANCH for southern Dade County in 1990 are shown in figure 17. A very good match between observed and simulated stages can be seen at the upstream S-338 and S-331 sites. However, these sites are relatively close to the northern boundary of L-31N Canal (fig. 4) where stage is defined in the model with no intervening structures. Therefore, the close match can be attributed primarily to boundary effects. Observed and simulated stages at northeastern canal sites S-120, S-122, and S-123 also closely match, which probably is attributable to the proximity of ground-water boundaries. Simulated stages at coastal structures S-21 and S-21A were lower than observed values. This discrepancy may be due to more computed leakage to tide than is realistic.

Observed discharges at canal structure sites and simulated canal discharges produced by BRANCH for southern Dade County in 1990 are shown in figure 18. These flows are strictly controlled by the hydraulic structures. The model computes structure operations with defined operating rules based on upstream stage criteria. In reality, these operating rules are not always followed due to manual operating decisions and other various overriding concerns. It was necessary, therefore, to change some of the operating rules from their official form in order for the operations in the model to match those in the actual system. In addition, an option was added to the Structures subroutine so that if a pump were pumping within the initial conditions, this pumpage would continue at the structure at the initial rate even if the initial stages were below the specified pumping limit. Once the upstream stage first exceeds the critical stage, all subsequent pumpages are determined by the operating rules. This procedure was devised in order to represent S-331, which, in figure 18, is seen to be pumping at the beginning of 1990 even though the stage is below the design criteria of 5 ft above sea level. All other initial conditions at structures are assumed (according to the model) to be reflected by the initial flow and stages.

Ground-water heads simulated by MODFLOW during 1990 are shown in figure 19. Comparison with the canal stages shown in figure 17 indicate a close correlation between surface-water levels and ground-water heads. Simulated water levels in the canals are similar to heads in adjacent ground-water wells, such as at paired structure S-338 and well G-855. Where the canal water levels tend to be simulated low, such as at structure S-21A, ground-water heads also tend to be low, such as at well G-1183 (fig. 19).

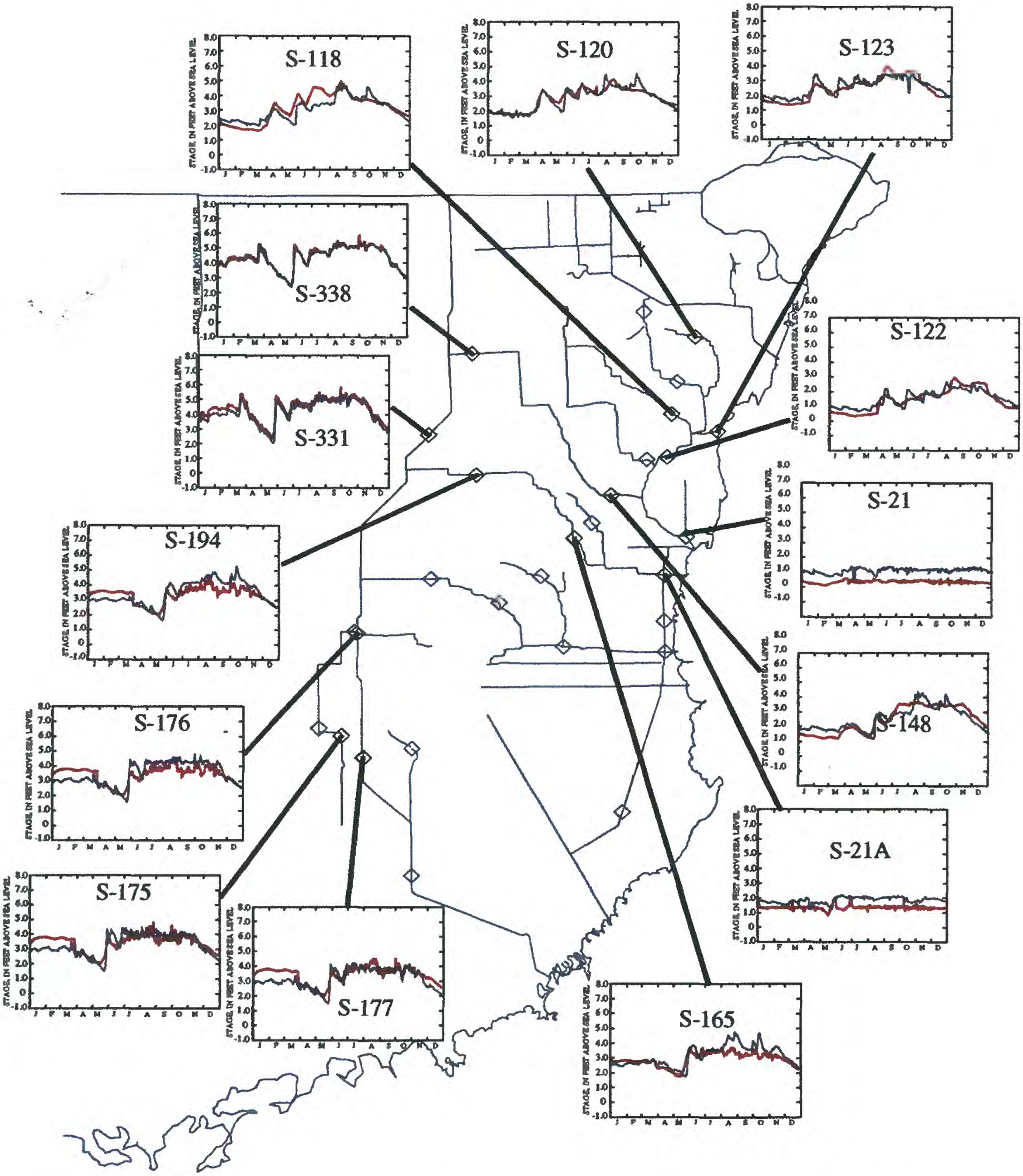

Figure 17. Observed (blue line) and simulated (red line) canal stages (BRANCH) for southern Dade County in 1990.

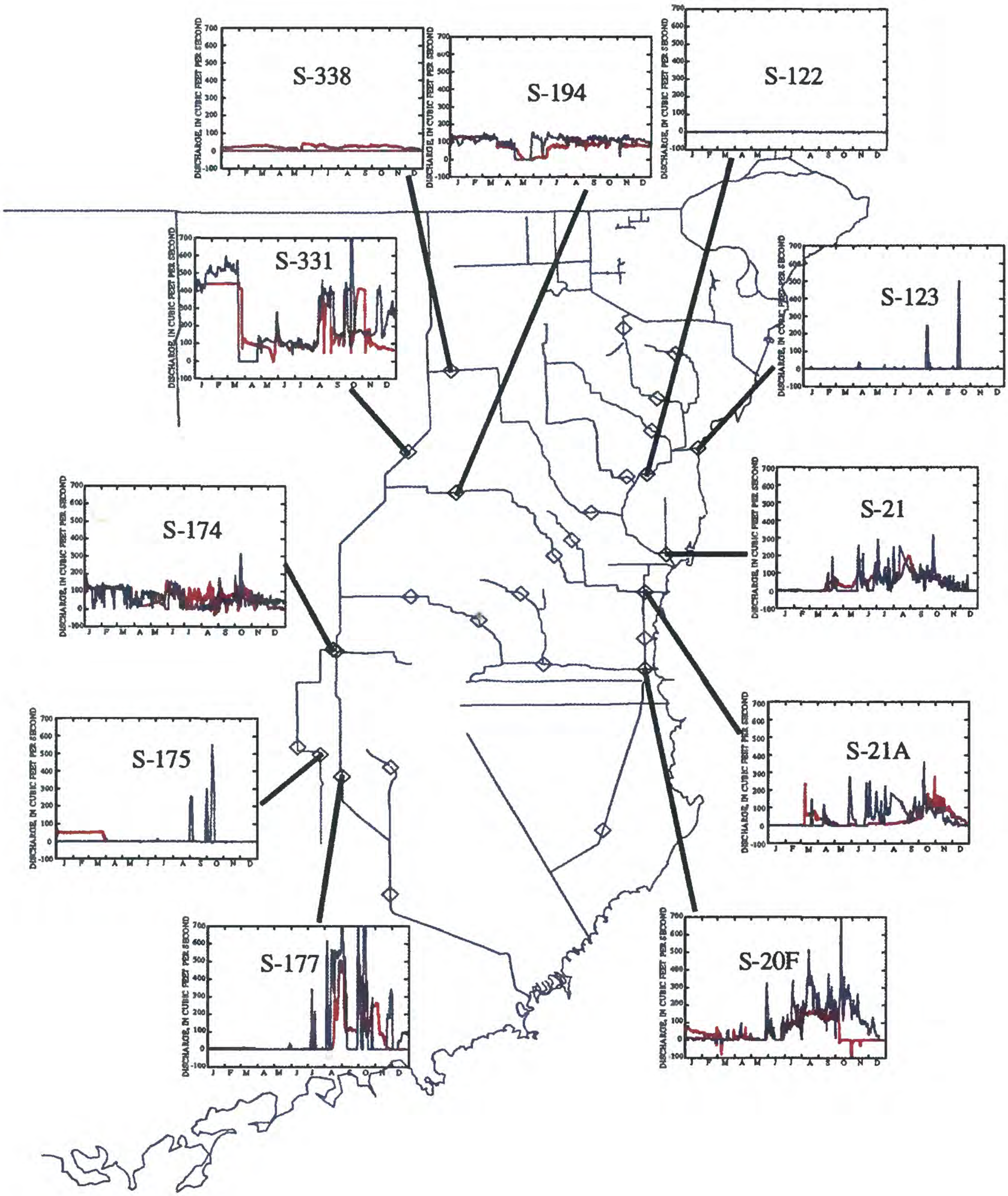

Figure 18. Observed (blue line) and simulated (red line) canal discharges (BRANCH) for southern Dade County in 1990.

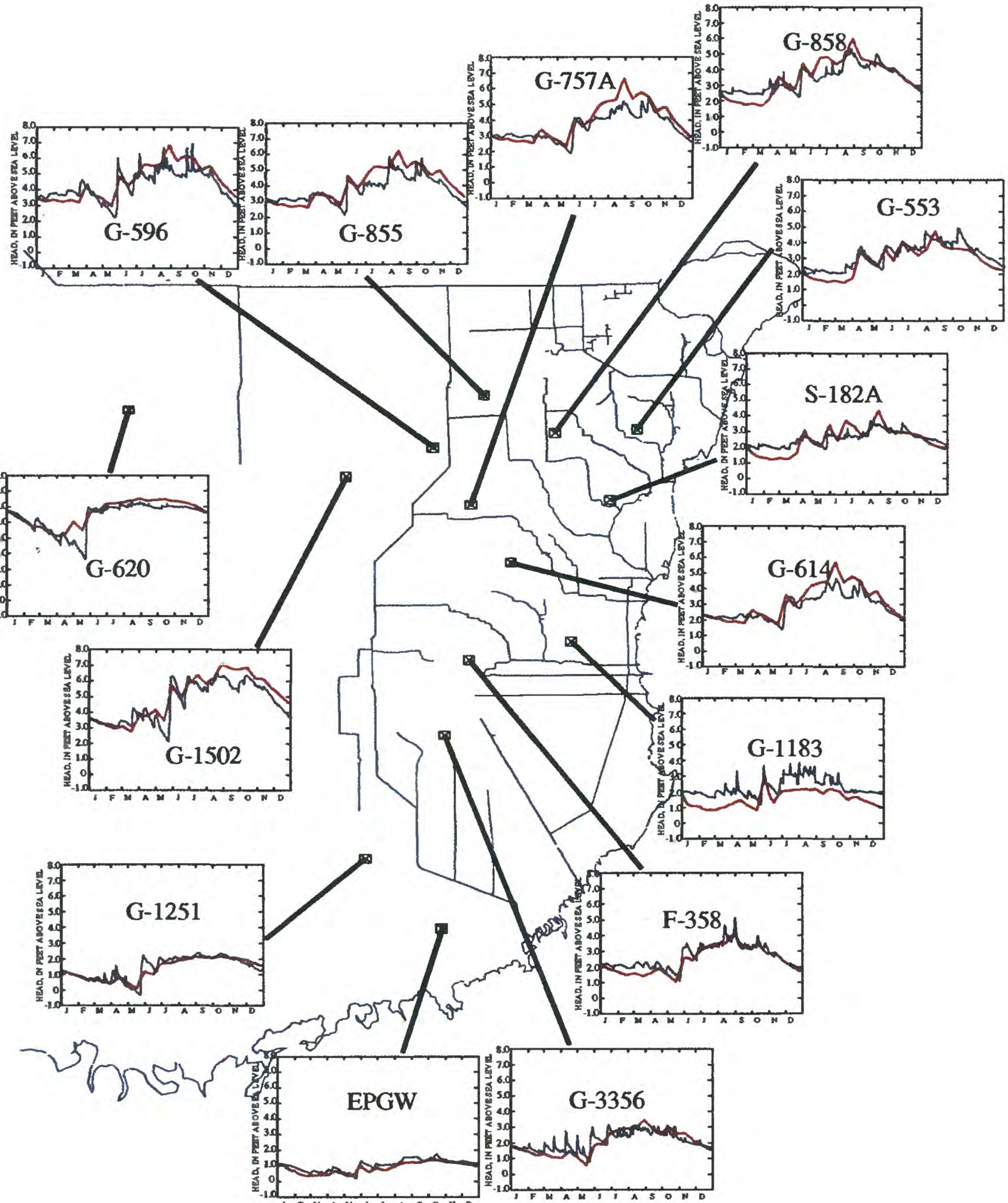

Figure 19. Observed (blue line) and simulated (red line) ground-water heads (MODFLOW using MODBRANCH) for southern Dade County in 1990.

Water levels in the wetlands area (layer 1 of MODFLOW) are shown in figure 20. Most of these sites are farther from the canals than the ground-water wells and are less affected by canal stage. The matches with field data are reasonably close, indicating that wetlands stages can be reasonably simulated by MODFLOW.

Cycles of wetting and drying were computed at a prodigious rate by the BCF2 package in MODFLOW during the simulation of the 1990 calendar year. The inundated areas simulated by the model at 30-day intervals are shown in figure 21. The areas that are wet in layer 1 (overland flow) are shown as the darkest zones. Simulated drying continues through the first half of April and terminates at the beginning of the wet season in the second half of May. Significant receding of the inundated areas does not occur until November. Simulated conditions at the end of the 1990 calendar year are wetter than at the beginning of the year. Model results of this type could be used to determine hydroperiods for various wetlands areas if model input data are available at the required scale.

Sensitivity Analysis

To evaluate the fit of the calibration and to determine the sensitivity of the model to changes in various input parameters, a root-mean square error analysis was done at each of the calibration points (figs. 17-20). The mean square difference between the observed and simulated values is computed for each site. A good calibration fit should have a small value of the mean of all these values. Also, the standard deviation of errors of all these site values should be small since outliers are undesirable. Although no attempt is made to optimize the calibration (being beyond the scope of this study), this analysis allows a quantification of the model fit to measured data and demonstrates the accuracy of the model. The sensitivity of the model can be quantified by the change in the root mean square error when input parameters are varied.

The input parameters selected for sensitivity analysis are those which are known with the least certainty. Evapotranspiration, lateral hydraulic conductivity of the aquifer, and Manning's n in the canals are principal parameters for sensitivity analysis. The output parameters for analysis are the canal stages and discharges produced by BRANCH and the aquifer (including layer 1 wetlands area) heads produced by MODFLOW.

The input parameters identified above were varied and the corresponding results are given in table 5. The first entry ("none" under parameter varied) corresponds to the calibrated condition. Comparison to the other results when parameters are varied indicates that no variation in any single one of the selected parameters causes an improvement in the overall results. However, some computed results are improved slightly by varying a given parameter. In general, computed results do not seem to be especially sensitive to the parameters selected, which are probably due to the highly controlled nature of the system. Surface-water hydraulic structures maintain water levels in the canal system with a direct effect on ground-water levels. Raising or lowering the BRANCH boundary stages by 0.5 ft causes the mean errors in canal stages to change less than 0.1 ft. This minimal difference is due to the flow controlling effect of the hydraulic structures.

Comparison of MODBRANCH and River Package Results

MODFLOW was initially created with the River package (RIV1), which represents the effects of surface-water channels having a user-defined stage (McDonald and Harbaugh, 1988). Leakage between these surface-water channels and the aquifer is represented by the same leakage equation used in MODBRANCH. RIV1 does not model flow or stage in the channels, but maintains the user input stage in the channel and assumes the channel can supply or receive as much water as is needed to maintain this stage. The stage can be varied every stress period, which is a defined set of timesteps in MODFLOW.

Because RIV1 is a simpler method for representing canals in hydraulic contact with an aquifer, it is of interest to compare RIV1 results with the MODBRANCH results for Dade County. Data for RIV1 were developed for all the canals represented by MODBRANCH and substituted for MODBRANCH in the southern Dade County model. Canal stages were input as monthly averages, and the same leakage coefficients were used as in the calibrated MODBRANCH model. All other data for the model were retained.

RIV1 does not compute stages or discharges in the canals, but heads and water levels computed for the aquifer and wetlands using only MODFLOW are shown in figures 22 and 23, respectively. Differences with the MODBRANCH results (figs. 19 and 20) are most visible at ground-water stations G-596, G-855,

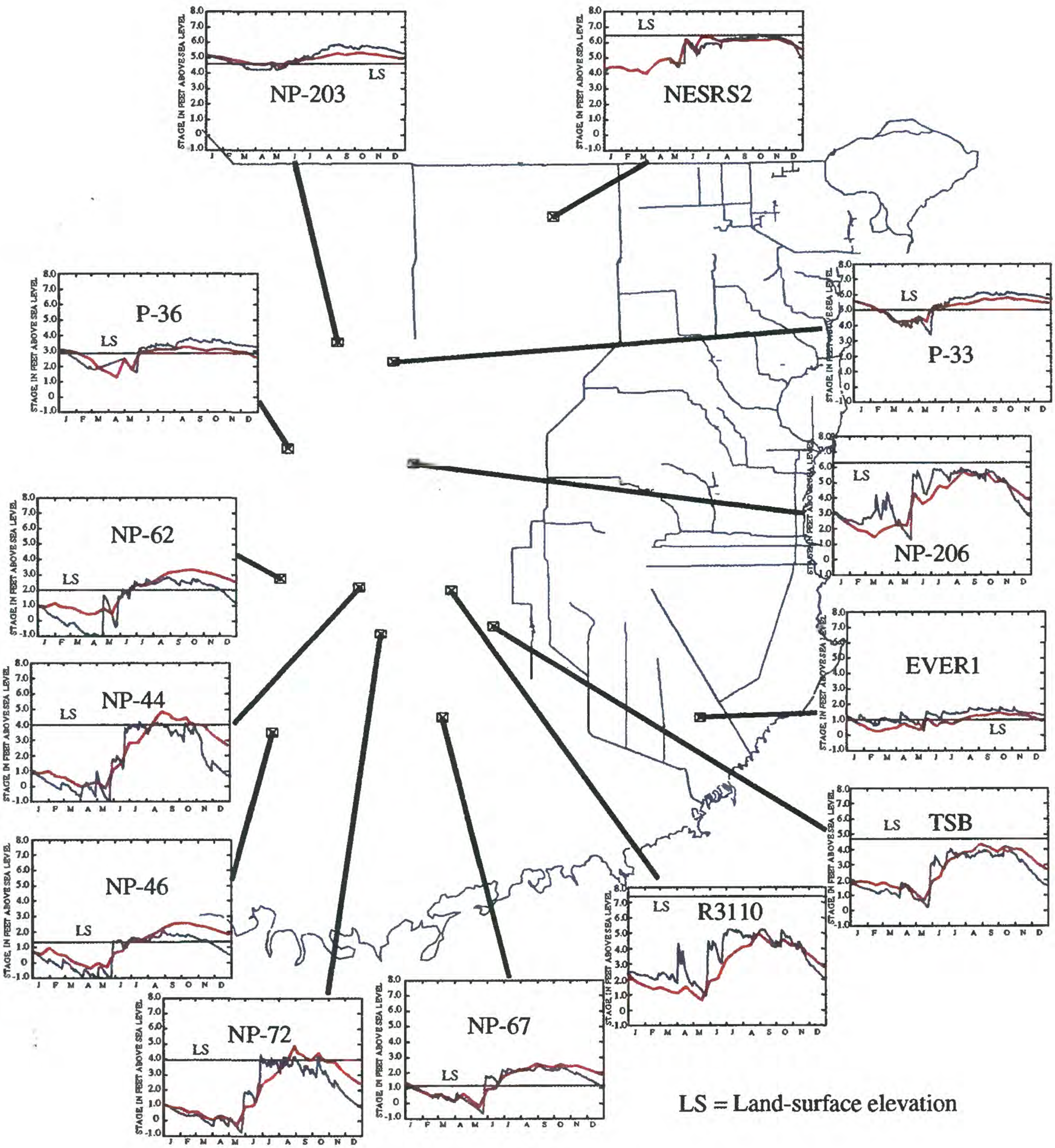

Figure 20. Observed (blue line) and simulated (red line) wetlands water levels (MODFLOW using MODBRANCH) for southern Dade County in 1990.

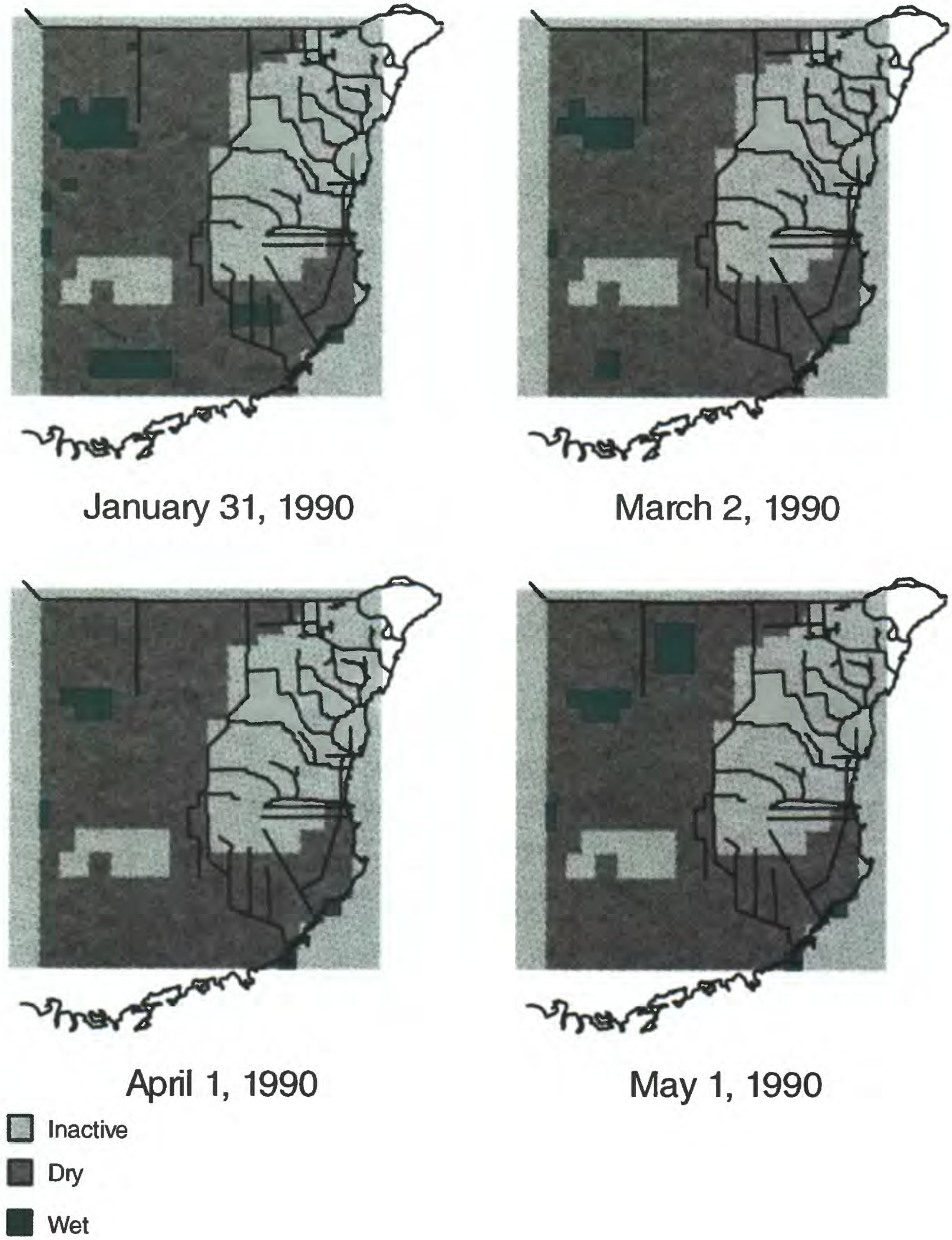

Figure 21. Inundated areas of wetlands simulated by the southern Dade County model.

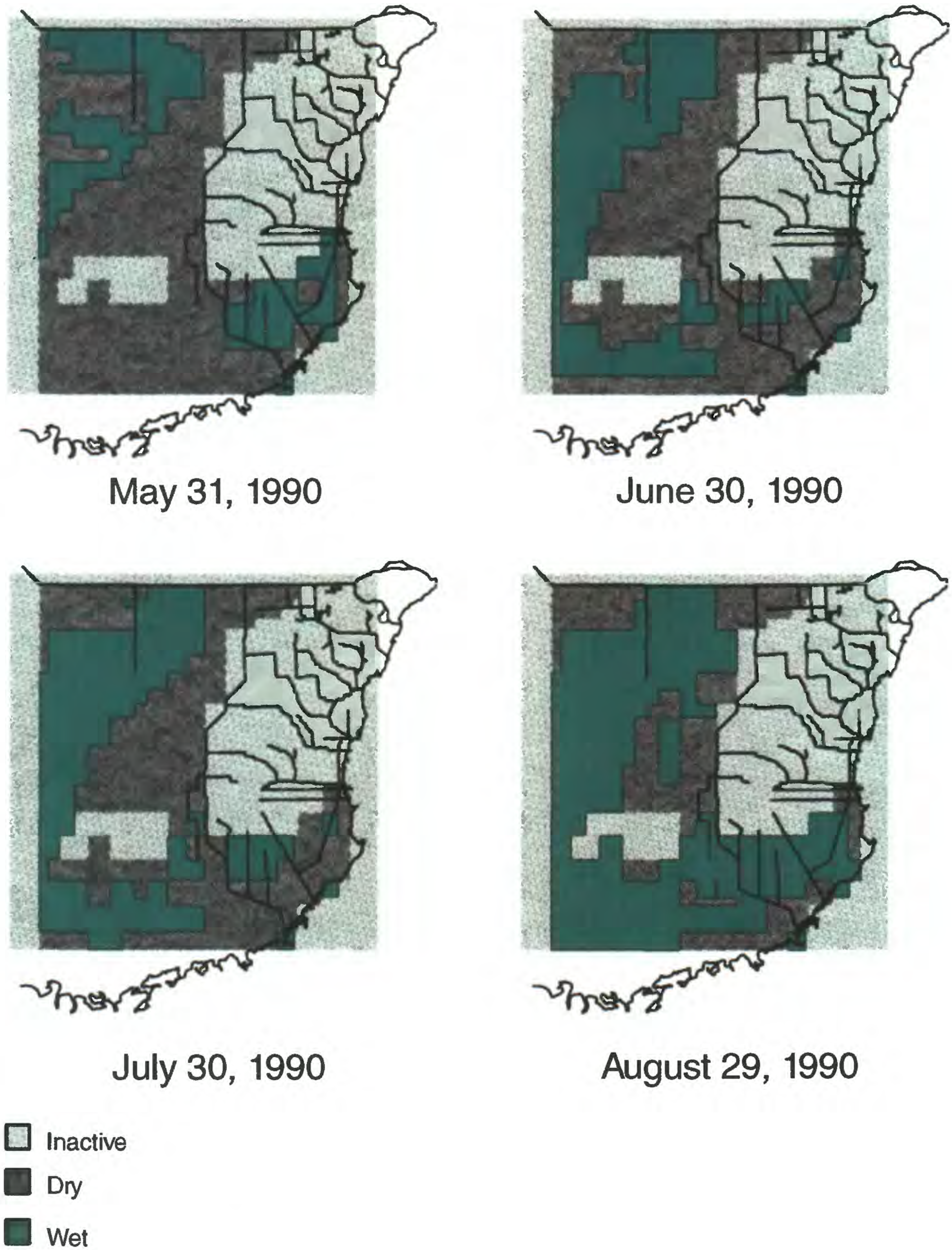

Figure 21. Inundated areas of wetlands simulated by the southern Dade County model--Continued.

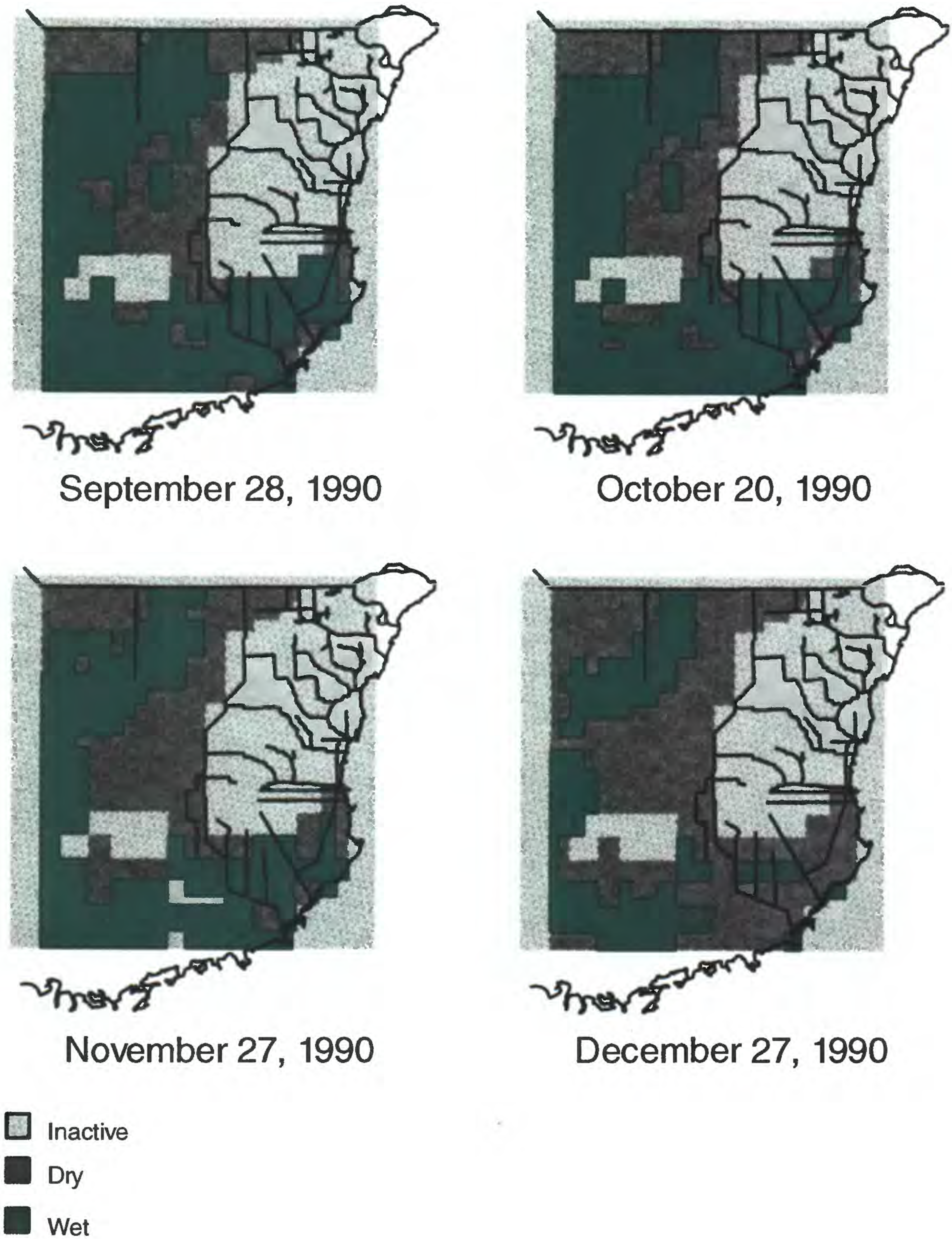

Figure 21. Inundated areas of wetlands simulated by the southern Dade County model--Continued.

Table 5. Results of the southern Dade County model sensitivity analysis

[Dashes indicate not applicable]

Parameter varied	BRANCH stages (feet)		BRANCH discharges (cubic feet per second)		MODFLOW heads (feet)	
	Mean	Standard deviation of errors	Mean	Standard deviation of errors	Mean	Standard deviation of errors
None	0.315	0.104	48.3	33.6	0.247	0.093
Evapotranspiration						
Raised 10 percent	.322	.105	49.1	34.4	.256	.099
Lowered 10 percent	.320	.104	46.8	32.4	.275	.123
Canal - Manning's *n*						
Raised 10 percent	.315	.105	46.6	31.4	.261	.105
Lowered 10 percent	.321	.105	47.7	30.9	.262	.105
Leakage coefficient						
Raised 10 percent	.315	.103	47.7	31.3	.260	.104
Lowered 10 percent	.322	.107	47.4	31.9	.263	.106
Boundary stages in BRANCH						
Raised 0.5 foot	.359	.096	54.4	44.9	.266	.110
Lowered 0.5 foot	.403	.140	56.7	53.3	.263	.101
Hydraulic conductivity		2				
Raised 10 percent	.312	.107	47.9	32.9	.257	.104
Lowered 10 percent	.318	.107	46.9	32.5	.267	.107
RIV1 used instead of MODBRANCH	--	--	--	--	.286	.124

G-757A, and S-182A. Significantly, these sites are near canals. The statistical analyses for sensitivity are given in the last row of table 5. The mean and standard deviation of the errors using RIV1 are larger than those using MODBRANCH. The main cause of this is probably the fact that when calculating leakage, RIV1 must maintain the same user-defined canal stage over a stress period; whereas in MODBRANCH, the stage is computed every surface-water timestep. This, coupled with the fact that MODBRANCH actually represents dynamic flow conditions in the channel, indicates the advantage of MODBRANCH as an alternative to RIV1 for complex flow conditions. However, the RIV1 data set is smaller and the run time of the model using RIV1 is on the order of 1/30th of that using MODBRANCH, indicating that RIV1 is less computationally demanding and may be preferable for application to simpler systems, for problems not requiring as much accuracy, and for situations in which canal flows are not of interest.

Wetlands Representation — SWIFT2D and MODFLOW Models

Although MODBRANCH represents one-dimensional surface-water flow in canals with the full dynamic flow equations, no presently available code exists to represent the two-dimensional overland flow as anything but Darcian flow through a highly conductive upper layer of the aquifer. To compare this method for representing overland flow in wetlands to the two-dimensional, fully dynamic flow equations, the southeastern coastal area (fig. 24) was modeled using the SWIFT2D model. SWIFT2D solves the finite-difference forms of the vertically integrated equations of mass and momentum conservation in two dimensions (Leendertse, 1987). This model is commonly used for tidal studies of estuaries and estuarine wetlands, but it has no means of representing underlying ground water. A useful attribute of the SWIFT2D model is its capability to rewet cells that have become dry, which is analogous to the rewetting capability in the MODFLOW BCF2 package. By modeling a wetlands area

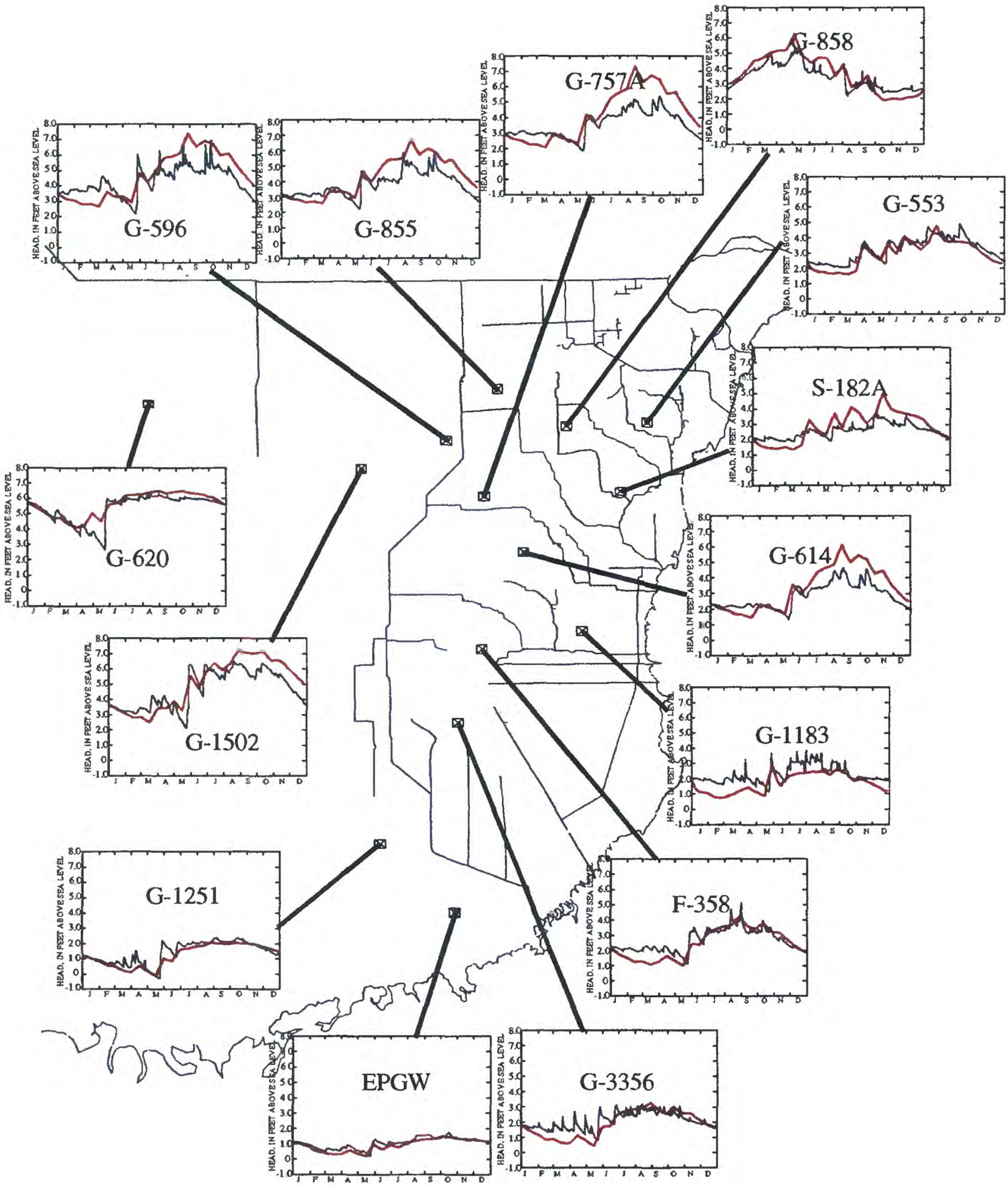

Figure 22. Observed (blue line) and simulated (red line) ground-water heads from MODFLOW using the River package for southern Dade County in 1990.

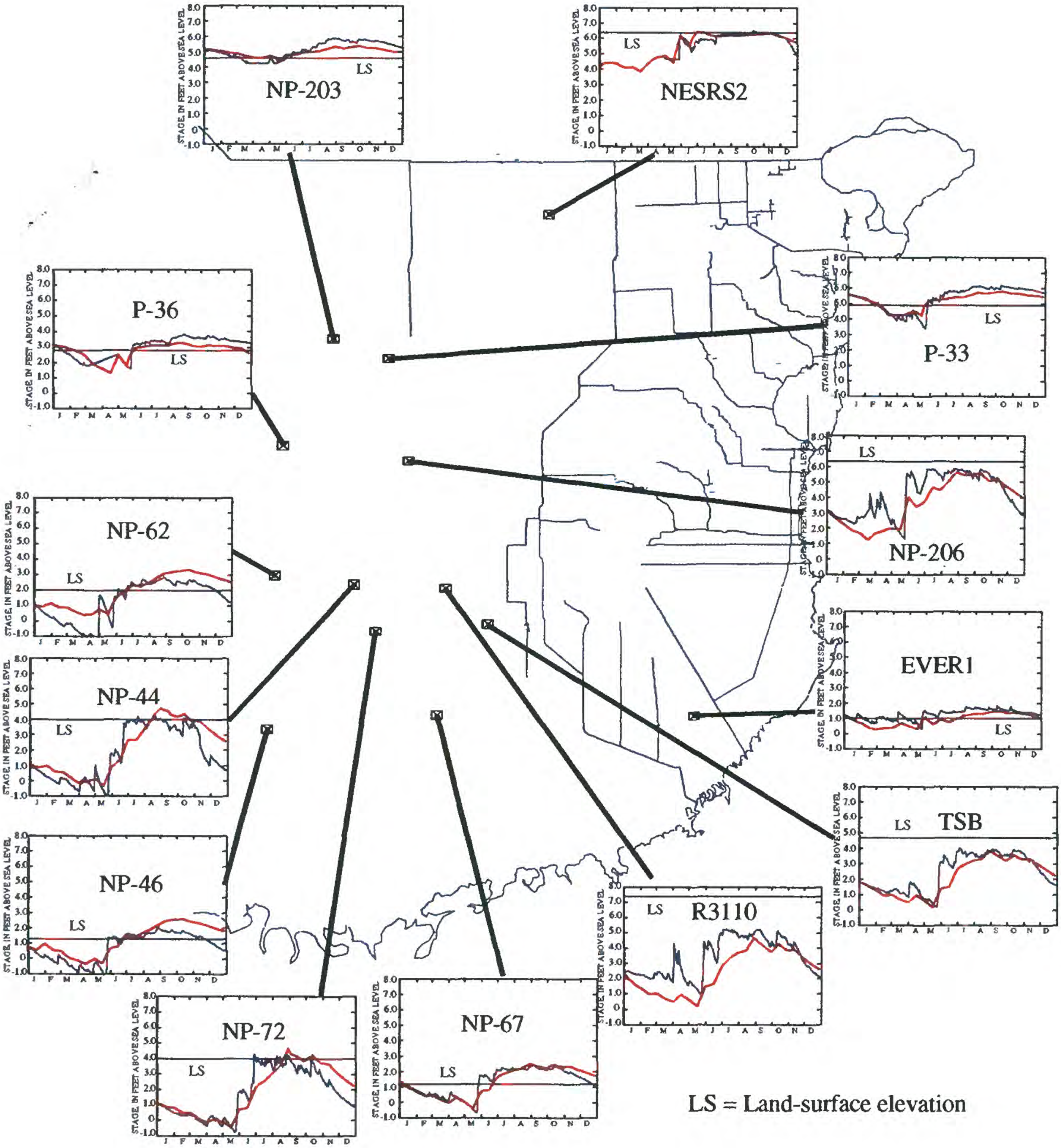

Figure 23. Observed (blue line) and simulated (red line) wetlands water levels from MODFLOW using the River package for southern Dade County in 1990.

both with the SWIFT2D and MODFLOW models, evaluations of the potential of each model can be made.

The area selected for this part of the study (fig. 24) contains intermittently inundated wetlands that are tidally affected some distance inland, thus making it an ideal area to test the SWIFT2D model. The section of the MODFLOW model grid that covers this part of the study area is shown in figure 25. Only the data in this section of the grid are used in the simulation run for SWIFT2D. Because the SWIFT2D model requires that the grid cells be square, it was decided that a 2,000 by 2,000 ft grid size be used (fig. 26). The fine grid consists of 39 rows and 37 columns. Cells that correspond to levees or other obstructions to overland flow are made inactive. Land-surface elevations for

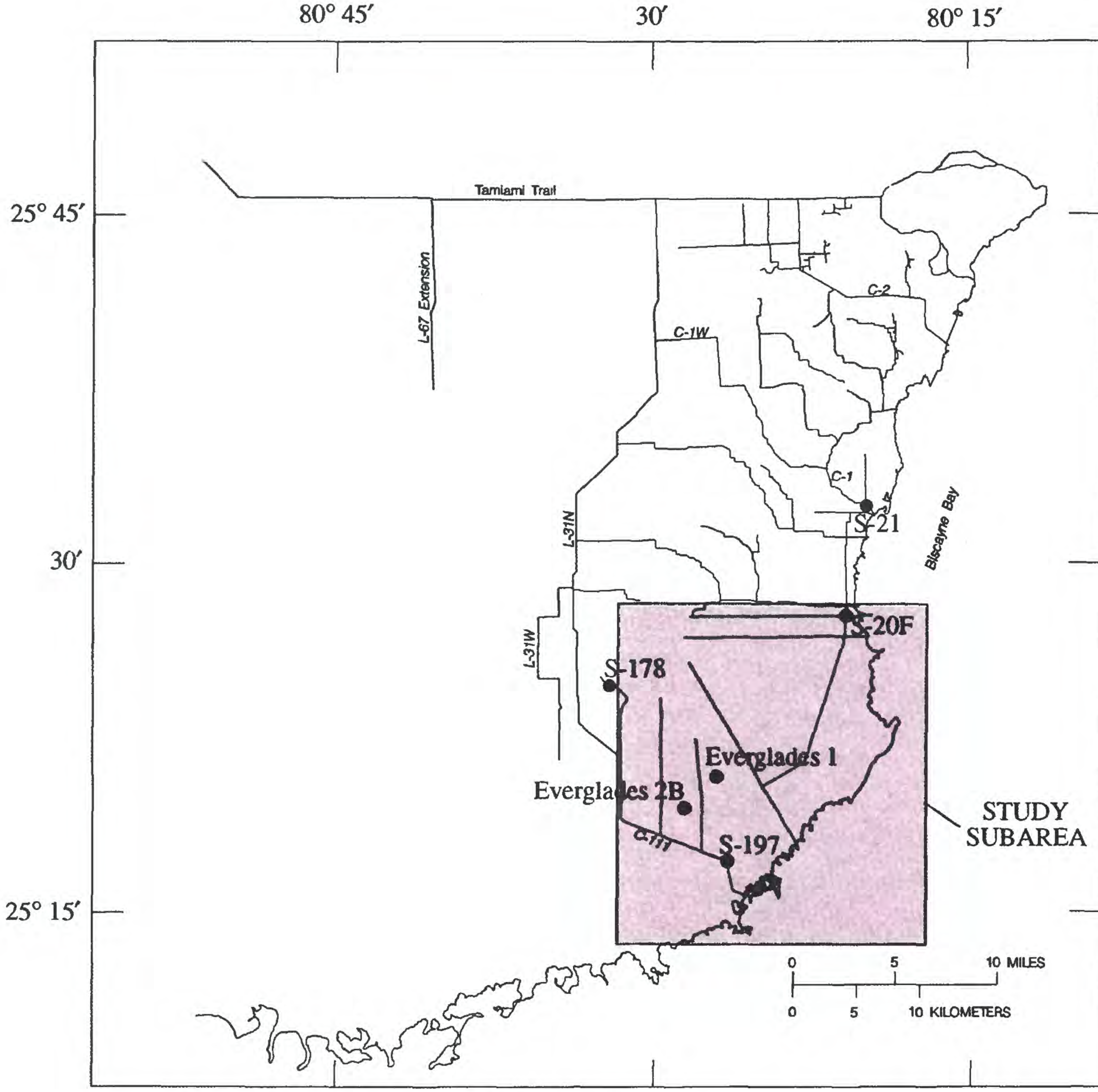

Figure 24. Study subarea for SWIFT2D/MODFLOW comparison.

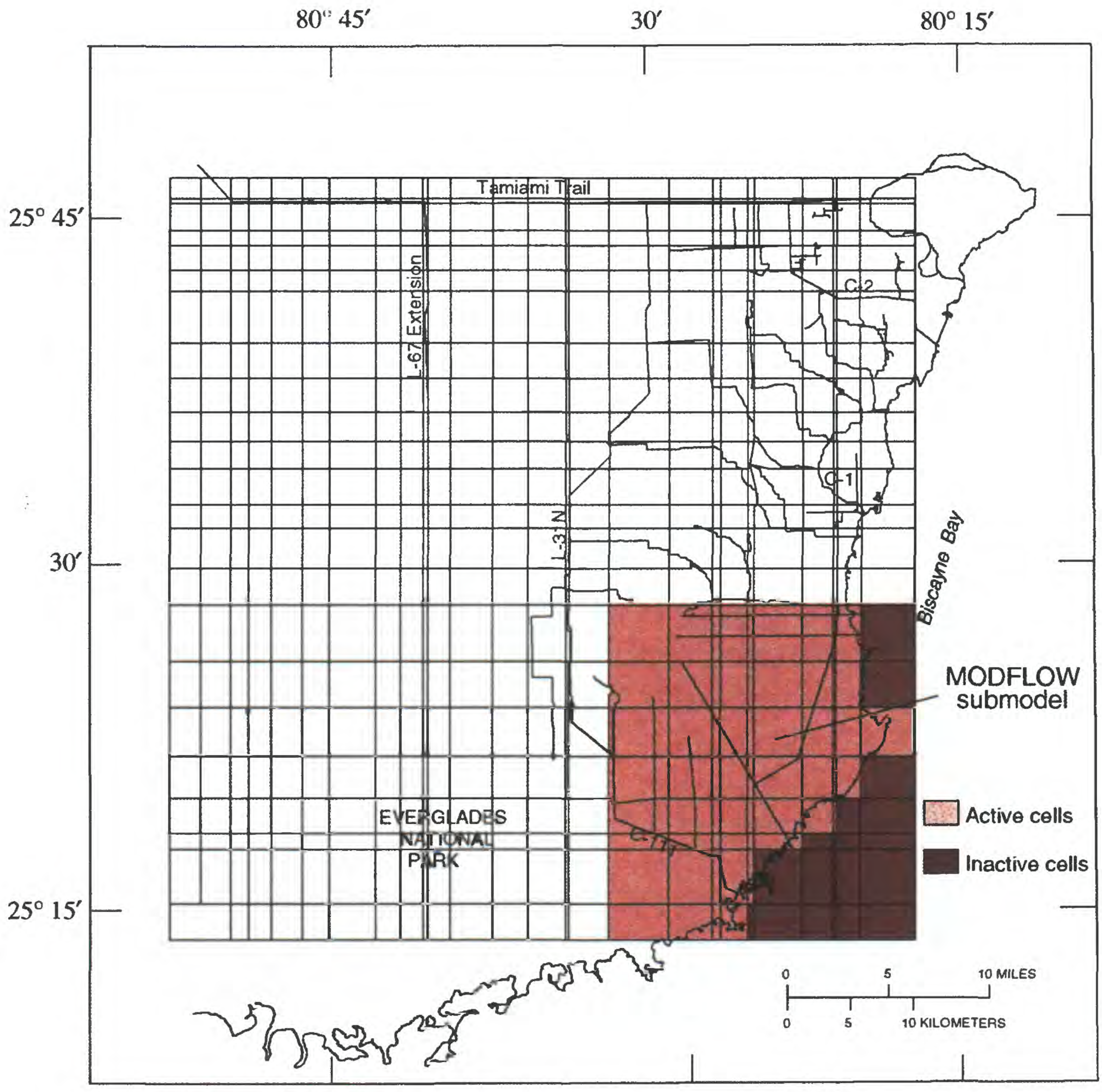

Figure 25. Section of Dade County MODFLOW model used to generate data sets for SWIFT2D/MODFLOW comparison.

each grid cell are interpolated from the data used in the southern Dade County model grid (fig. 25). A 15-minute timestep is used, corresponding to the frequency at which data are collected from the stage measurement sites.

The comparison period selected for the two models was August 1-7, 1988. This relatively wet period is ideal for evaluating overland flow. Boundary data were used from the four sites shown in figure 24 (downstream values at S-20F, S-21, S-178, and S-197). Water-level values are linearly interpolated along the boundaries to obtain intermediate values for all the boundary cells.

To ensure that a direct correspondence exists between the MODFLOW model and the SWIFT2D counterpart, the same cell size (2,000 by 2,000 ft) is used in the MODFLOW model. Parameters, such as vertical and lateral hydraulic conductivities, and layer

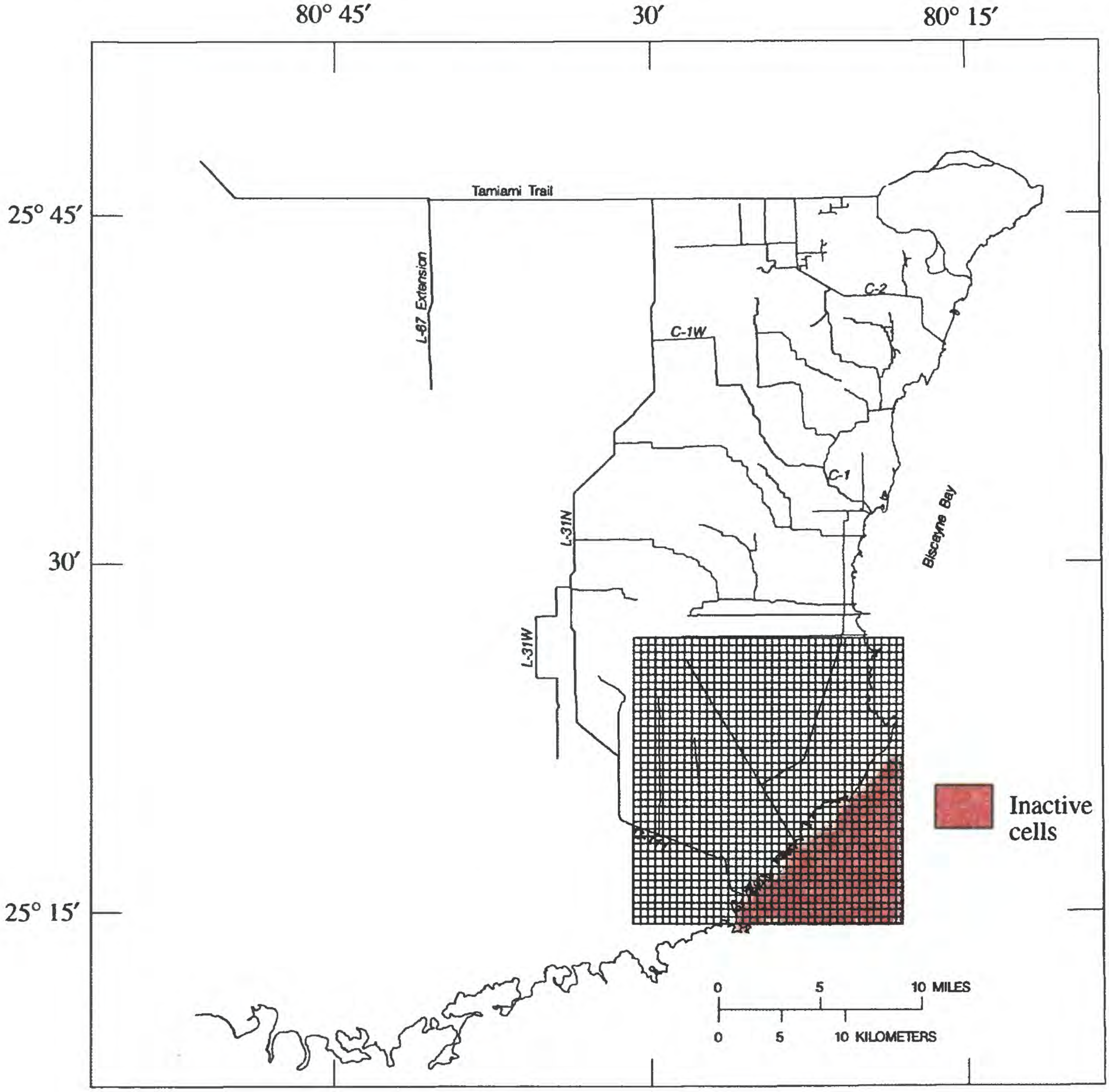

Figure 26. Model grid for the SWIFT2D/MODFLOW comparison.

elevations are maintained the same as for the southern Dade County model. As with the SWIFT2D model, a 15-minute timestep is used. To change the boundary conditions at 15-minute intervals, a new stress period in MODFLOW must be defined for each timestep. In this way, the general head boundaries defining the tidal boundaries are allowed to change at the same incremental timestep as the SWIFT2D model.

The frictional resistance to flow must be represented by equivalent factors in each model. This is not a simple transformation since MODFLOW represents flow by Darcy's law (where flow is proportional to slope) and SWIFT2D represents flow by solution of coupled equations of mass and momentum conservation in which important frictional resistance effects are approximated by a Manning's type expression (in which flow is proportional to the square root of slope). Merritt (1992) stated the relation between equivalent Manning's n and Darcy's K by:

$$n = \frac{1.486D^{2/3}}{KS^{1/2}} \tag{22}$$

where,

- n is Manning's n;
- D is the depth of flow, in feet;
- K is the lateral hydraulic conductivity, in feet per second; and
- S is the water slope.

The MODFLOW model uses an "equivalent hydraulic conductivity" for overland flow K of 30,000,000 ft/d = 347.2 ft/s (feet per second). The model shows an average depth, D, of about 1 ft and a slope, S, of 0.05 ft in 2,000 ft or 0.000025. Using these values in equation 22 would yield an equivalent Manning's n of 0.856. This value, although very large, is not unexpected for wetlands vegetation, and it was used in the SWIFT2D simulation.

The MODFLOW and SWIFT2D simulations are compared with hourly field measurements at stations Everglades 1 and 2B in the model area (fig. 24), and the results are shown in figures 27 and 28. At station Everglades 1, a small fluctuation is evident in SWIFT2D at twice the tidal cycle (fig. 27). This result is not reflected by MODFLOW nor by the field measurements. These fluctuations are small and could be the result of a numerical effect related only to the equations and their solution. The MODFLOW results are in closer agreement with field measured data at station Everglades 1 than SWIFT2D results. At station Everglades 2B, only daily mean measured stages are available. These are plotted as pulses in figure 28. At station Everglades 2B, the SWIFT2D model results are in closer agreement to the field measured data than the MODFLOW results (fig. 28).

The MODFLOW model predicts lower water levels at both stations. The SWIFT2D and MODFLOW models do not predict as high a water-level gradient between stations Everglades 1 and 2B as is shown by the field measurements, indicating that the resistance to flow could be higher in actuality than that assumed. Also, the fluctuations in water level shown by SWIFT2D at station Everglades 1 are not reflected by MODFLOW nor by the field measurements. This could suggest that fluctuations are dampened by leakage to or from the underlying aquifer, which is not accounted for by SWIFT2D.

APPLICATION OF A SUBMODEL WITH MODIFICATIONS FOR STRUCTURES AND WETLANDS

A study of modified water deliveries to ENP (U.S. Army Corps of Engineers, 1992) describes alternatives for restoring more natural water levels to the Everglades and associated areas. Consequently, the effects on other parts of the system from raising water levels in the Everglades became of interest. The potential for an increase in flooding frequency, especially in an 8.5-mi^2 residential area where flooding is already a problem (fig. 29), required a modeling effort to predict the effects of remediation proposals.

The proposed modifications to the present hydrologic scheme are shown in figure 29. The L-67 Extension will be removed and the canal filled. A levee and canal stretching southwest from L-31N will be constructed to protect the 8.5-mi^2 residential area from higher water levels to the west and intercept ground-water leakage. Pump station S-357 will be placed where the residential canal meets L-31N to pump water from the residential canal into L-31N. Pump station S-356 will be placed on the former site of gated spillway S-334 to pump water west into the L-29 Canal. It is hoped that the S-356/S-357 pair will allow more water west in the L-29 Canal to be recirculated through the wetlands south of the canal. Structure G-211, which was made operational in 1991, regulates flow south in L-31N and can be used to ensure that flows from S-357 are diverted north to S-356. The two structures, S-355A and S-355B (fig. 29), are proposed gated spillways to release water from Water Conservation Area 3B (located north of the study area) into L-29 Canal. All these modifications to the system (U.S. Army Corps of Engineers, 1992) will result in significant changes to the entire hydrologic system of southern Dade County.

Of particular interest to the COE is the potential for flooding and flood mitigation in the 8.5-mi^2 residential area (fig. 29). Pump S-357 is the primary mechanism that would be used to remove excess water from this area, and pump S-356 must also be adequate to divert this additional flow to the west. To analyze the operation of all the proposed structures and to determine the necessary capacity of the pumps, a

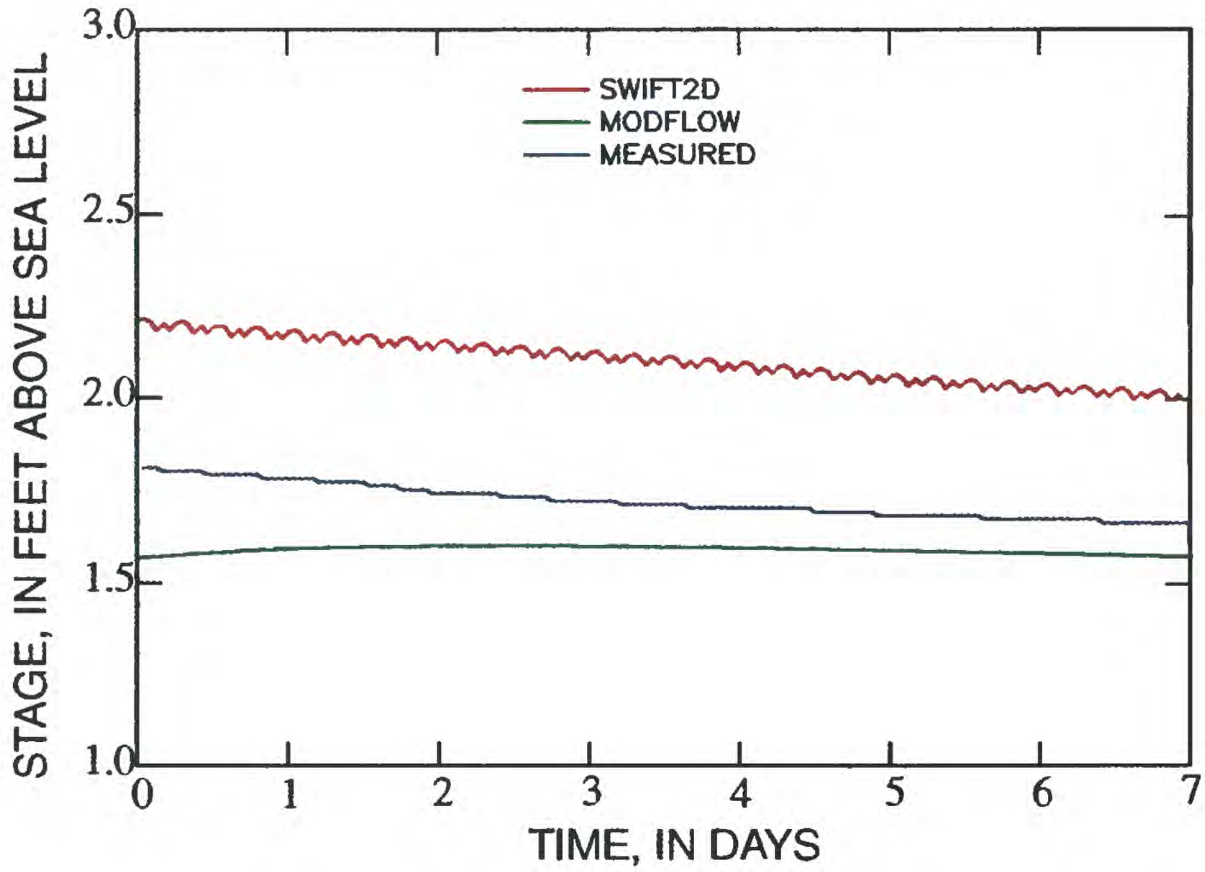

Figure 27. Observed and simulated stages at station Everglades 1.

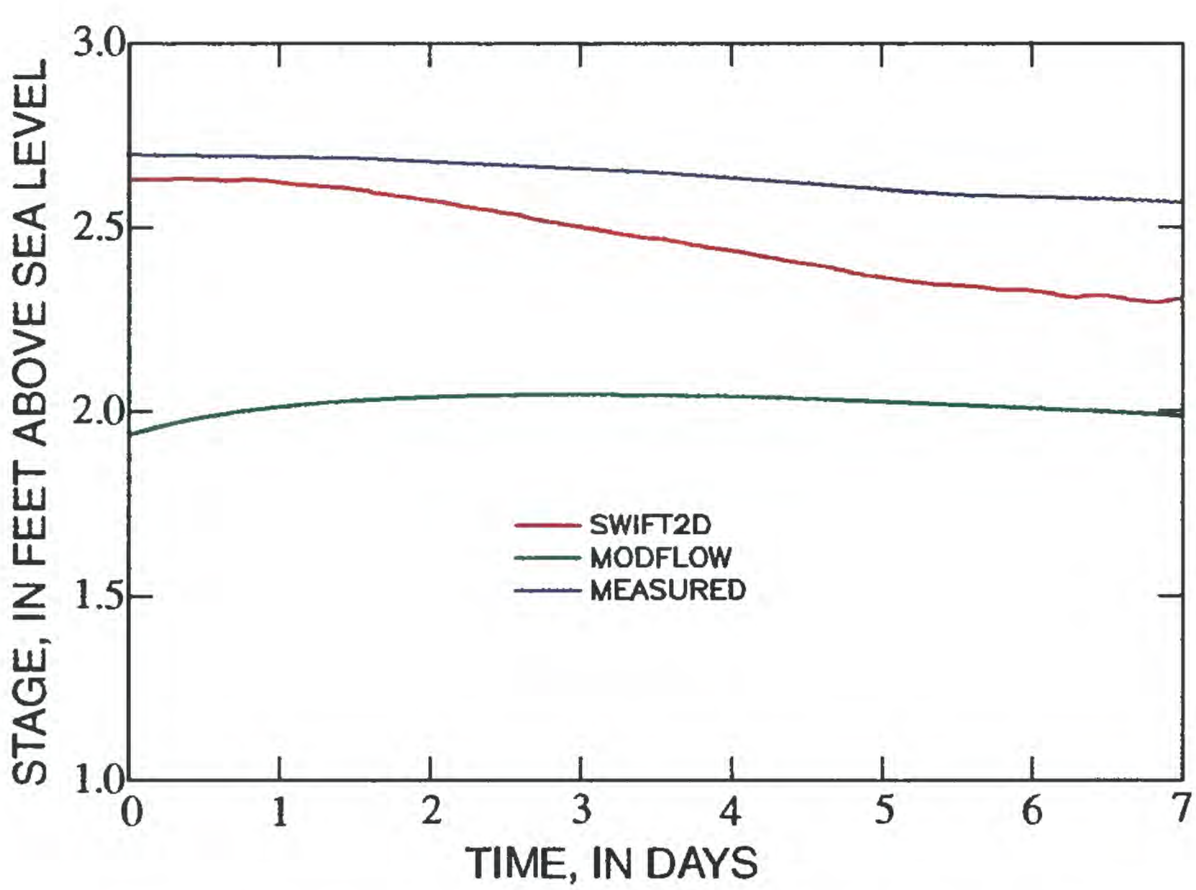

Figure 28. Observed and simulated stages at station Everglades 2B.

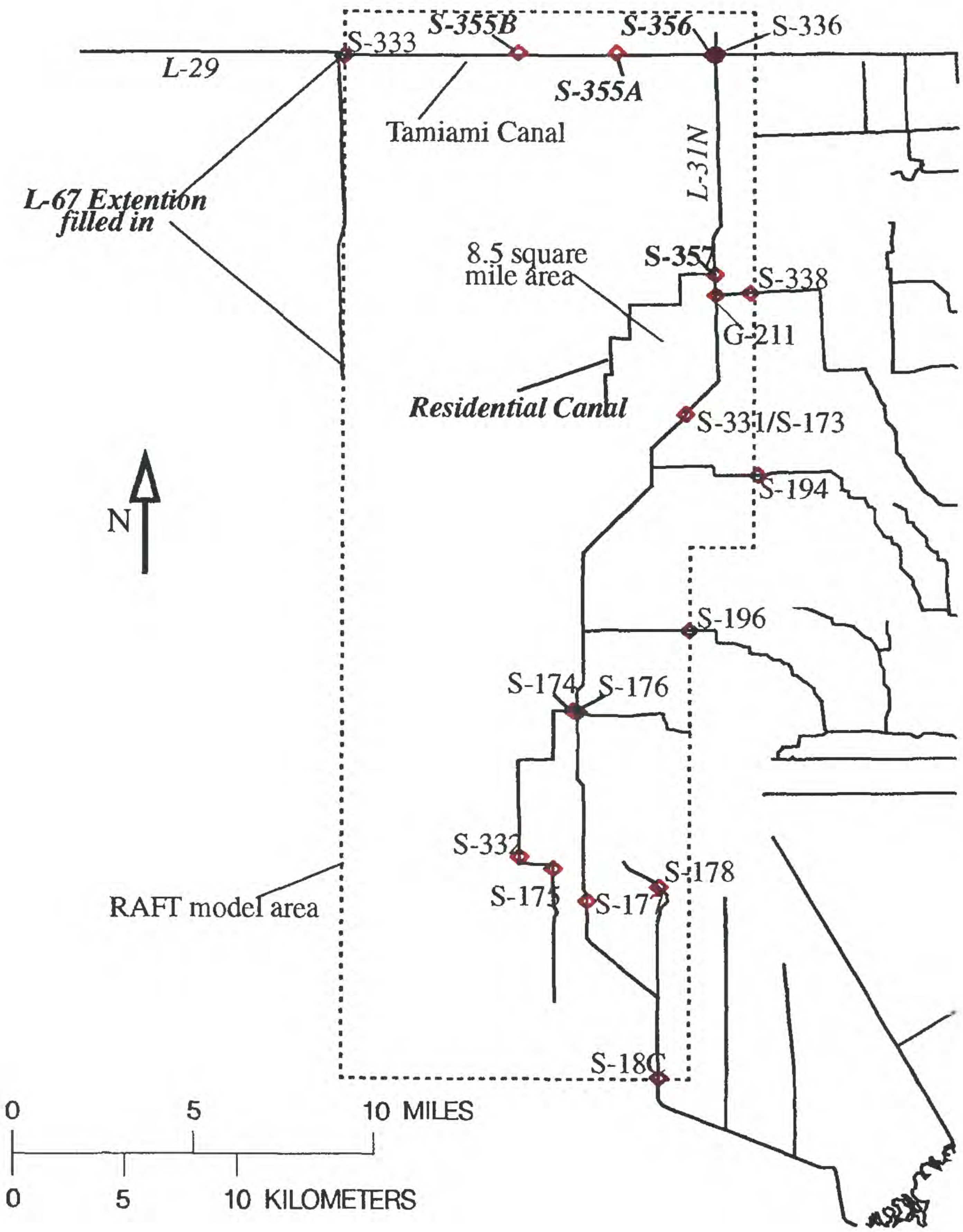

Figure 29. RAFT model area and proposed modifications to system (proposed modifications in bold).

model was developed using the MODBRANCH package with the options as described earlier. This Residential Area Flood Test (RAFT) model covers a subarea of the southern Dade County model (fig. 29) and was selected to cover the area of interest with appropriate definable boundaries. The surface-water model has control structures at all locations where canals meet the boundaries. Water levels and discharges are measured at all these structures. For the MODFLOW model, the L-29 levee provides the northern boundary, the eastern boundary is defined with water levels interpolated between surface-water control structures, and the southern and western boundaries by ground-water and surface-water level monitoring stations.

The MODFLOW model grid for the RAFT model (fig. 30) is based on the southern Dade County model grid, but a finer grid spacing is used around the 8.5-mi^2 residential area. The inactive cells (hatch pattern) were selected to place the model boundaries at locations with known canal water levels. The RAFT model contains three layers, as in the case of the southern Dade County model, with the topmost layer representing overland flow.

The BRANCH model network contains 37 branches and 121 segments. Unlike the southern Dade County model, the Tamiami Canal is modeled in BRANCH so that the effects of structures S-355A, S-355B, and S-356 can be simulated. Ten structures are defined within the model area: S-356, S-357, G-211, S-331, S-173, S-174, S-175, S-176, S-177, and S-178 (fig. 29). Nine more structures define the boundaries to the canal model (BRANCH): S-333, S-355A, S-355B, S-336, S-338, S-194, S-196, S-332, and S-18C (fig. 29).

Boundary conditions for the BRANCH model are set according to the best data available as in the southern Dade model. The western boundary of Tamiami Canal is defined by daily discharges computed at S-333 by the USGS, and the eastern boundary of Tamiami Canal is defined by the mean daily discharge measurements made at S-336 by SFWMD. The boundary at the northern end of L-31N (where it connects to L-30 to the north) is defined by stages measured by the USGS. The actual location of this stage measurement is 1 mi south of Tamiami Canal, but it is considered representative of the stage in the area. The discharges at S-338 computed by SFWMD (zero flow for all of 1990) as well as stages at S-194 and discharges at S-196 were used to define that boundary. Discharge measurements made by the USGS for S-18C and those made by SFWMD for S-332 were used. The northern end of L-67 Extension Canal is defined by stage values collected by the USGS. The remaining dead-end channels, C-113, C-111E, and L-31W (fig. 4), were defined as no-flow boundaries.

As in the case of the southern Dade County model, the structure operating rules were either obtained from SFWMD design memoranda or estimated from field discharge measurements. Leakage coefficients for the canals were used as a calibration parameter and were different in value from the southern Dade County model because of the different size model cells in the two models (see appendix). Manning's n coefficients were set to a nominal value of 0.027.

The same calibration period, the 1990 calendar year, was selected as for the southern Dade County model. For this simulation, the structures not existing in 1990 (S-355A, S-355B, S-356, S-357, and G-211) were not included. No flow was allowed at S-357. The residential levee cells in MODFLOW were made active, and the leakage coefficient for the residential canal was set to 0 (so the residential canal and levee effectively did not exist).

The results of the calibration run are shown in figures 31, 32, and 33. The canal stages shown in figure 31 are for the same sites shown for the southern Dade County model (fig. 17) that are within the RAFT model area. In addition, because L-29 Canal is also included in the RAFT model, comparison of stages at Bridge 53 on L-29 is also included. Comparison of this simulation (fig. 33) with figure 17 shows a better match to observed canal stages than corresponding simulation by the southern Dade County model. This result can be attributed to the closer proximity of boundaries in the smaller RAFT model. The canal discharges in figure 32 compare equally well to those simulated by the southern Dade County model (fig. 18). The observed and simulated ground-water heads and wetlands stages are shown for the RAFT model in figure 33. All four stations in the model area show close comparison of simulated results to the observed values.

An initial test at a flood scenario was simulated with this model. Rainfall from 1969, a wet year, was substituted for the 1990 rainfall to simulate a flood condition. All boundary heads in MODFLOW were raised 2 ft. A constant flow rate of 400 ft^3/s (cubic feet per second) is input to L-31N from the north and

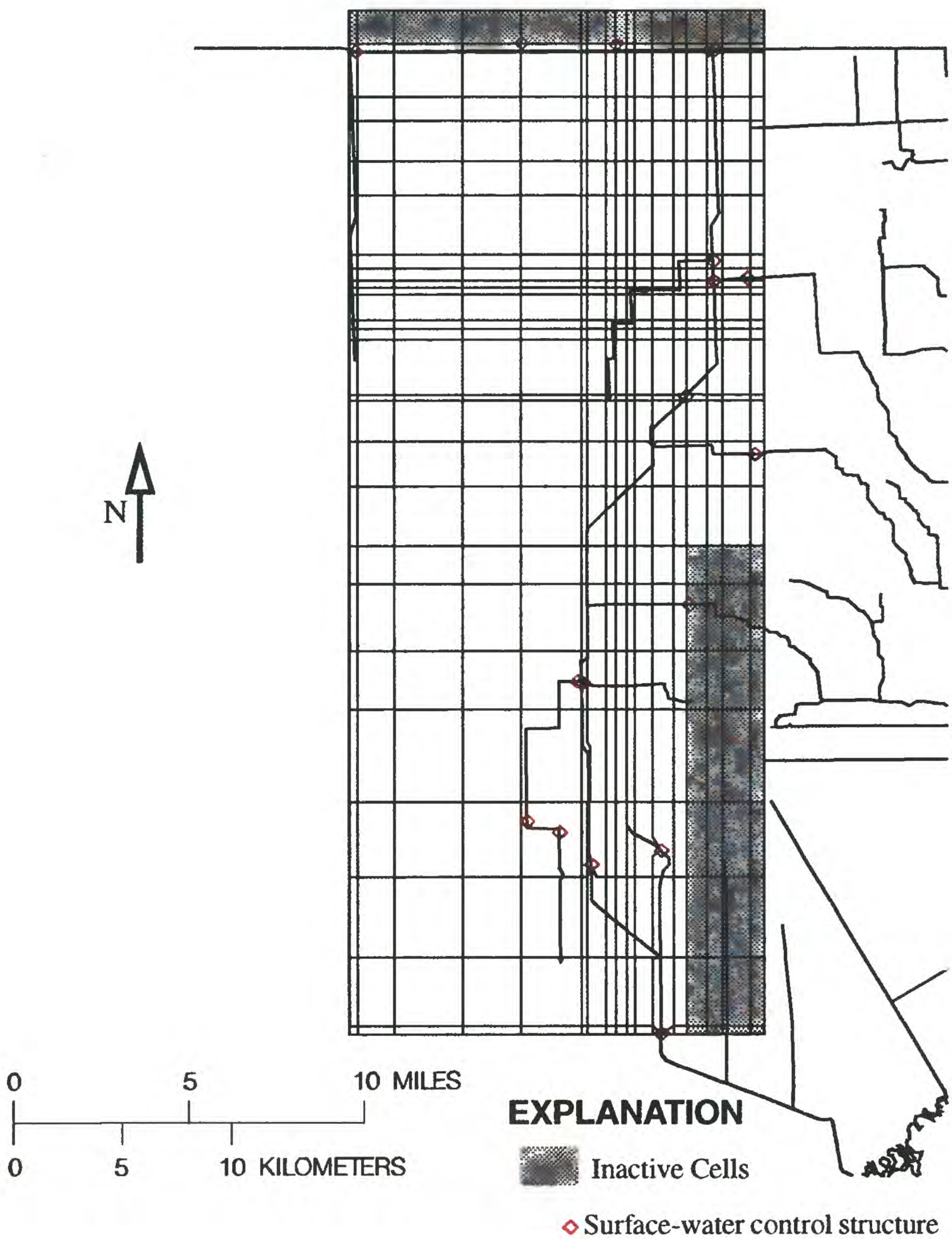

Figure 30. MODFLOW model grid for the RAFT model.

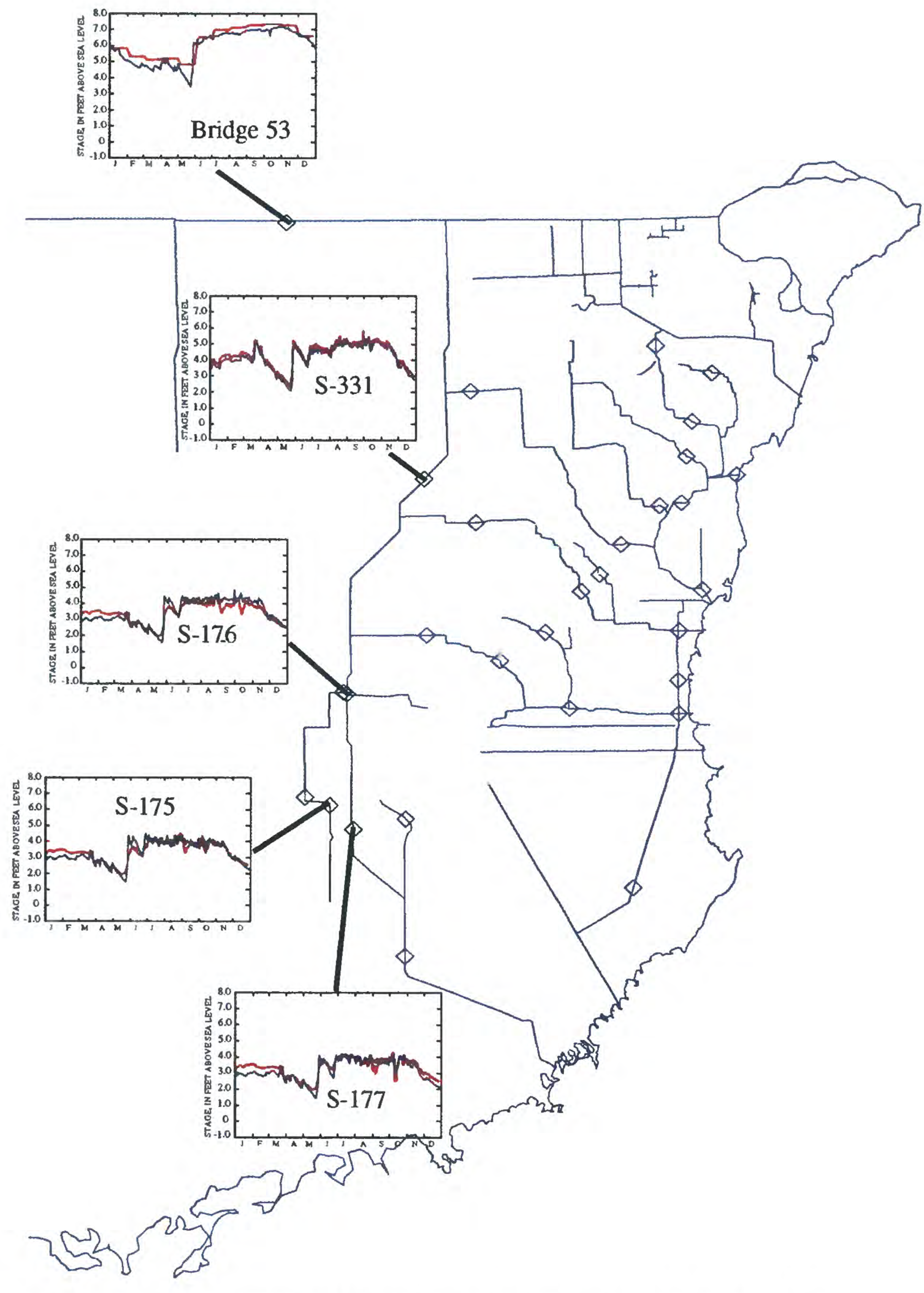

Figure 31. Observed (blue line) and RAFT model simulated (red line) canal stages for 1990.

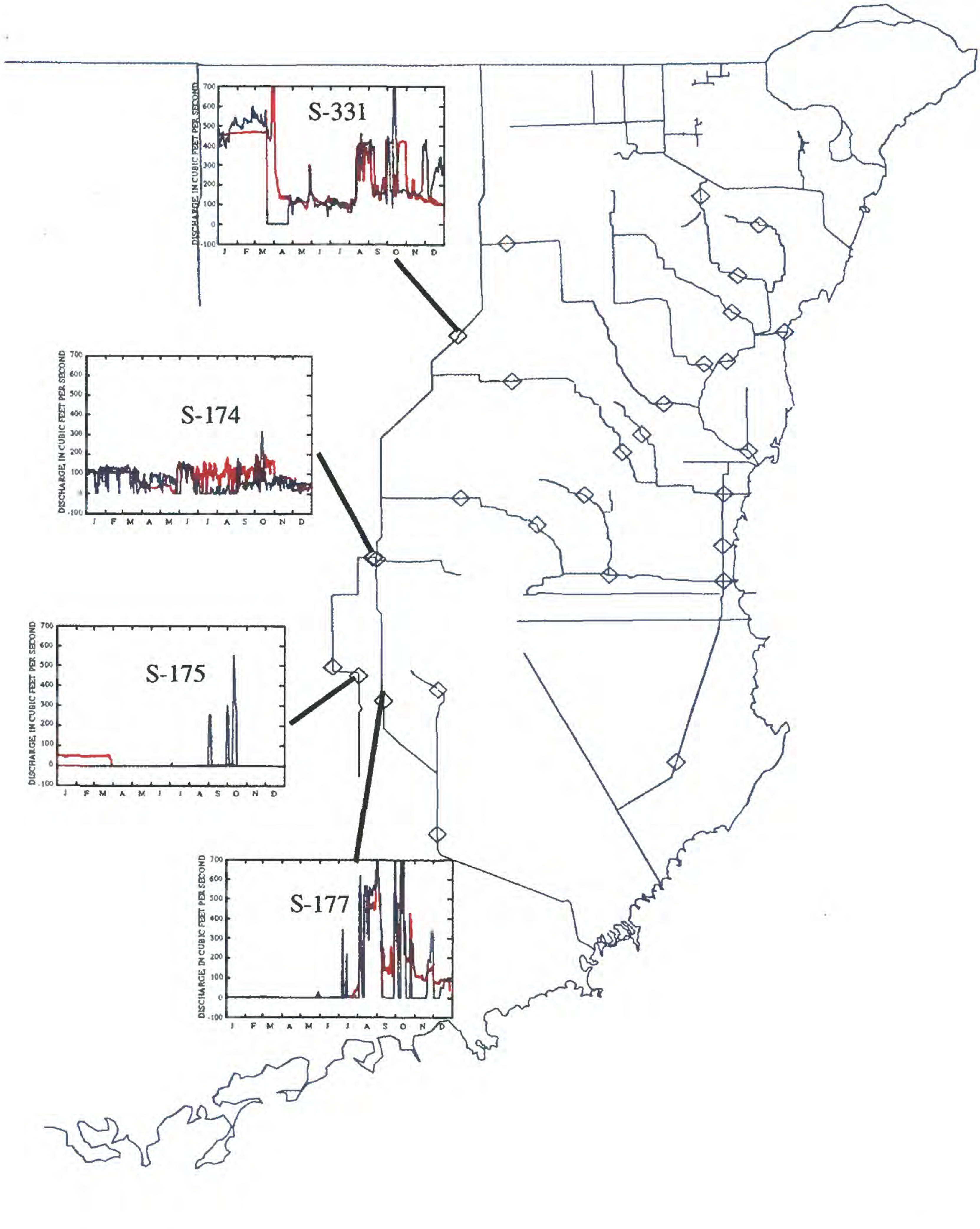

Figure 32. Observed (blue line) and RAFT model simulated (red line) canal discharges for 1990.

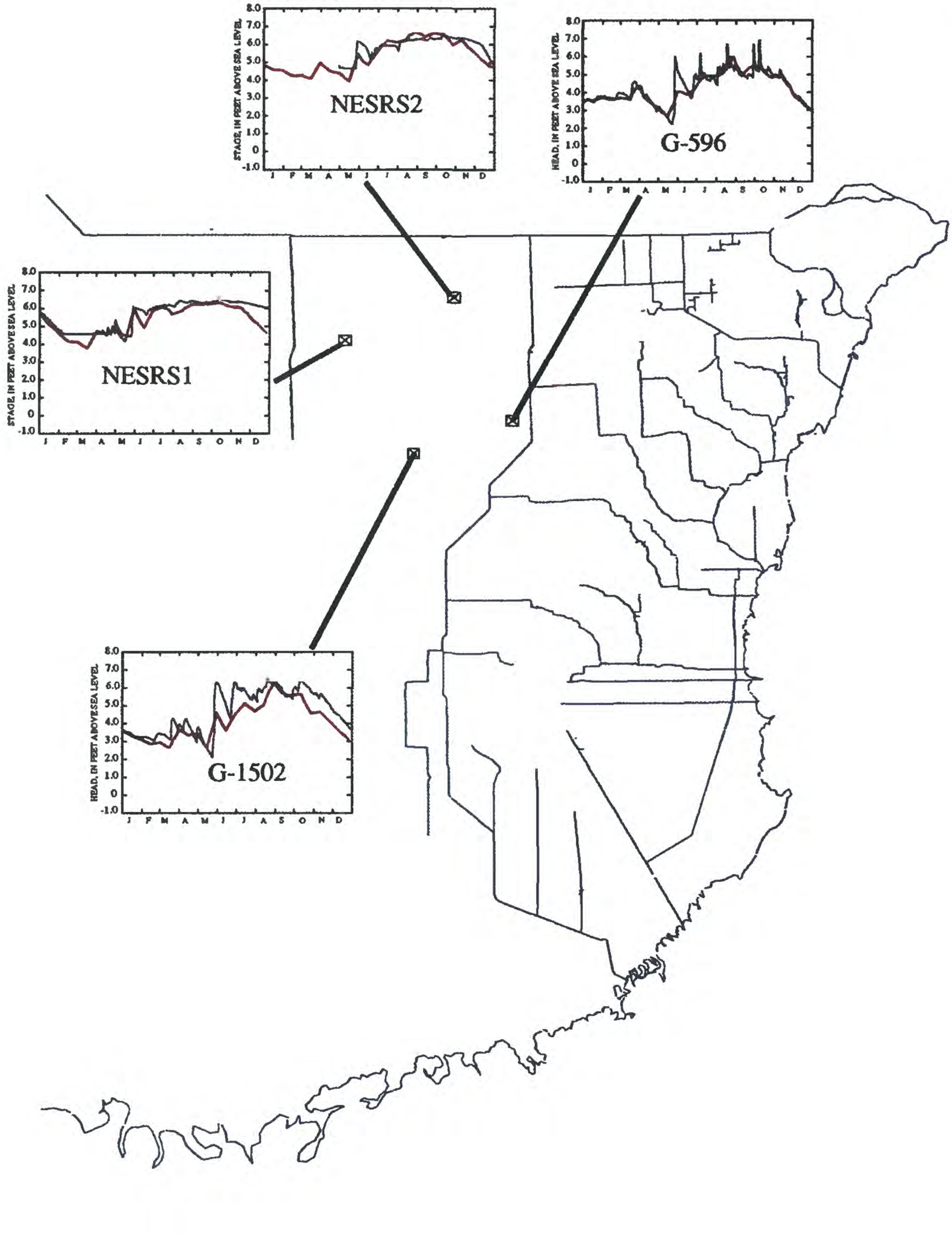

Figure 33. Observed (blue line) and RAFT model simulated (red line) ground-water heads and wetlands stages for 1990.

through the boundary structure S-336 (fig. 29). All other canal boundaries are set to no-flow conditions. Point inflows into L-29 Canal (250 ft^3/s each) are introduced to represent S-355A and S-355B. Structures G-211, S-356, and S-357 are placed into the model and the residential canal is represented with leakage. The results are shown in figures 34, 35, and 36. Stations S-356 and S-357 were set to operate when canal stages (fig. 34) reached 5.5 and 5.0 ft above sea level, respectively (both pumps were given infinite capacity to maintain these heads). Simulated discharges at stations S-356 and S-357 approached 400 and 150 ft^3/s, respectively (fig. 35). Simulated ground-water heads were as high as 7 ft above sea level in the 8.5-mi^2 residential area (fig. 36, site G-596). Simulated discharges were found to be low compared to measured discharge in the canal network south of G-211, which is represented in the model as closed. The COE is refining the model parameters to be used in its final determination of the flood-control scenario.

SUMMARY AND CONCLUSIONS

A study was conducted to test the coupling of a ground-water flow model (MODFLOW) with a surface-water flow model (BRANCH) for simulating a wetlands/surface-water/ground-water system on a regional scale. Southern Dade County located in southeastern Florida is an ideal area for such a test because of the area's unique environment and hydrology. Tasks completed during the study were: (1) modifications to the MODFLOW and BRANCH models; (2) incorporation of hydraulic structures into the BRANCH model; (3) construction, calibration, and verification of the coupled MODFLOW/BRANCH model; and (4) comparison of methods for determining appropriate uses and areas of applicability for different types of representation including the SWIFT2D model.

Extensive modifications to the MODFLOW and BRANCH models were required to couple the two models and to represent the wetlands/surface-water/ground-water system of southern Dade County. Before the development of the model of southern Dade County, the MODFLOW package, MODBRANCH, was written to allow the MODFLOW ground-water model to be linked to the BRANCH surface-water model, and another MODFLOW package, Streamlink, was written to represent the direct connection of canals to wetlands.

As part of the development of the southern Dade County model, another MODFLOW package, BCF2, which allows model cells to alternate between wet and dry states, was modified to represent seasonal wetting and drying of the wetlands. Use of BCF2 in representing the wetlands requires redefinition of some of its input parameters. The lateral hydraulic conductivity of the overland flow layer must represent an equivalent resistance to flow for the wetlands. The representative porosity of the overland flow layer appropriately is 1. The MODFLOW evapotranspiration package (EVT2) was modified to allow evapotranspiration to occur in differing layers as wetting and drying occur. The original EVT1 package did not allow evapotranspiration losses to be removed from lower layers when upper layers became dry. However, the modified package, EVT2, does have this capability. The relative merits of different modular techniques were compared using two methodologies: wetlands cells in a coastal wetlands were represented with a highly conductive upper layer of the aquifer in MODFLOW, and with the SWIFT2D two-dimensional dynamic surface-water model.

The MODBRANCH package allows stream-aquifer leakage to be accounted for in both MODFLOW and BRANCH. A subroutine was developed for BRANCH to represent the hydraulic structures that regulate the canal system in Dade County. This subroutine represents gated spillways, gated culverts, and pumps with defined operating rules. The Streamlink package represents connections where the canals open into the wetlands.

The southern Dade County model simulations suggest that defining the proper operating rules for the hydraulic structures is very important in properly representing the system. The published operating rules are not always followed. Both surface-water and ground-water conditions were reproduced quite well in the model, with seasonal wetting and drying accurately represented in the wetlands. Mean errors of 0.315 ft for stage and 48.3 ft^3/s for discharge were seen for the canals and 0.247 ft for the ground-water heads. A sensitivity analysis indicated the model was highly sensitive to evapotranspiration and boundary variations, and no alternative simulation was found that was overall superior to the calibrated model.

An analysis of the most accurate methods to simulate canals and wetlands was conducted by modeling the same area and conditions using models other than the linked MODFLOW/BRANCH model. To determine the best method for representing canals, a comparison was made with a simulation using the RIV1 package instead of MODBRANCH. RIV1 does not represent canal flow, and the user must define the canal stages. RIV1 had a larger mean error of 0.286 ft in ground-water heads. To determine the best technique for representing wetlands, a SWIFT2D model

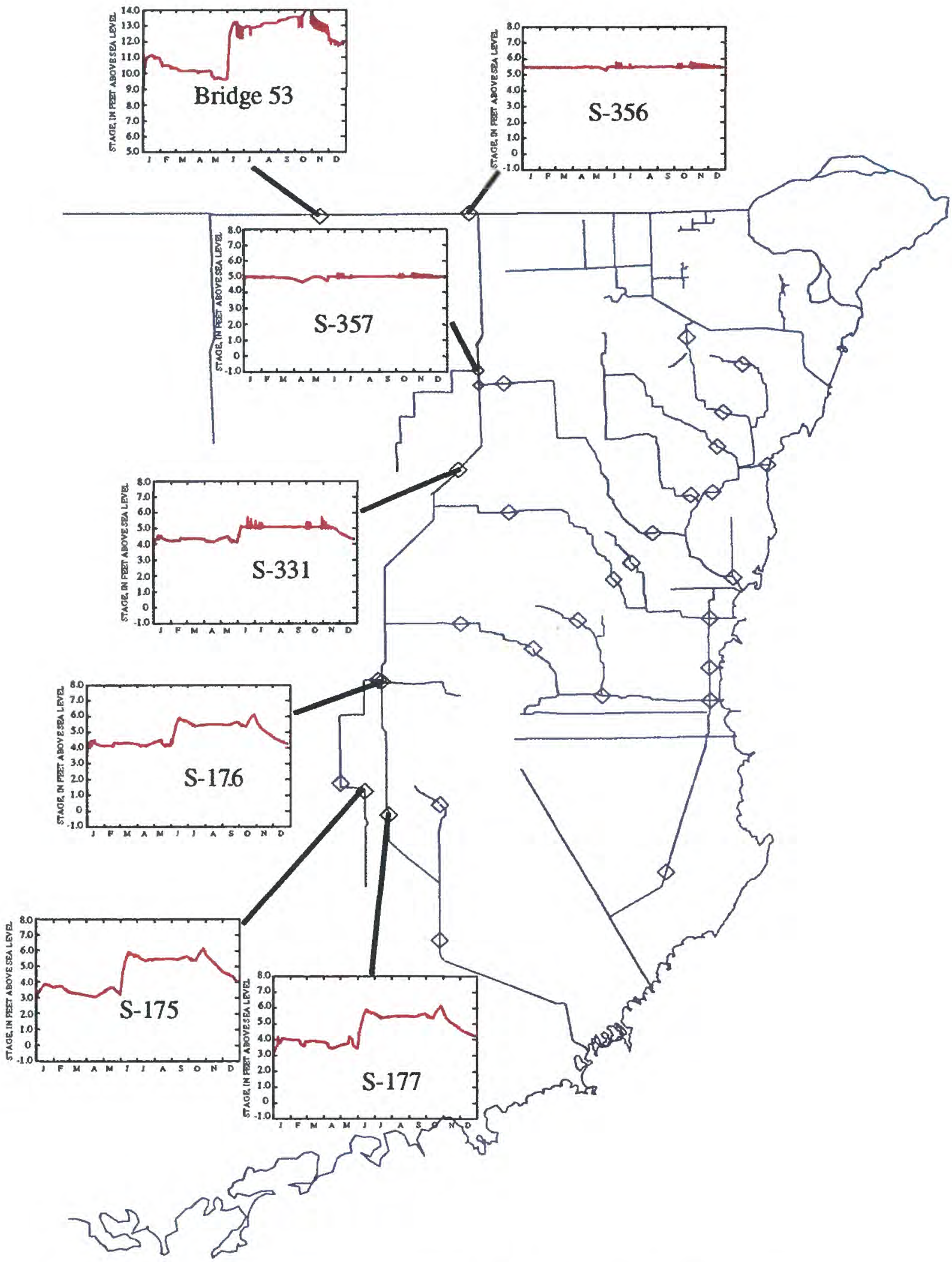

Figure 34. Canal stages (BRANCH) simulated by the RAFT model for flood scenario.

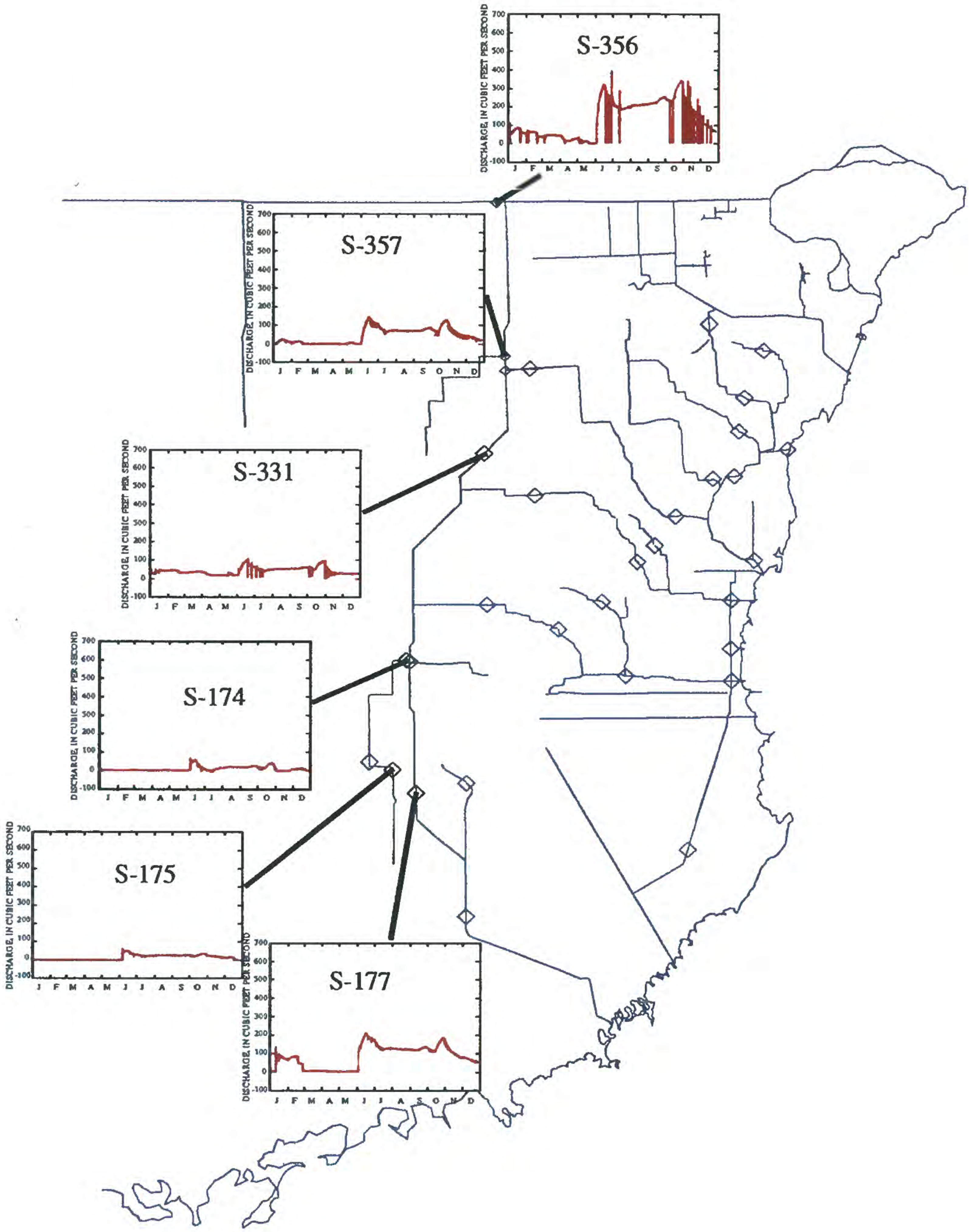

Figure 35. Canal discharges (BRANCH) simulated by the RAFT model for flood scenario.

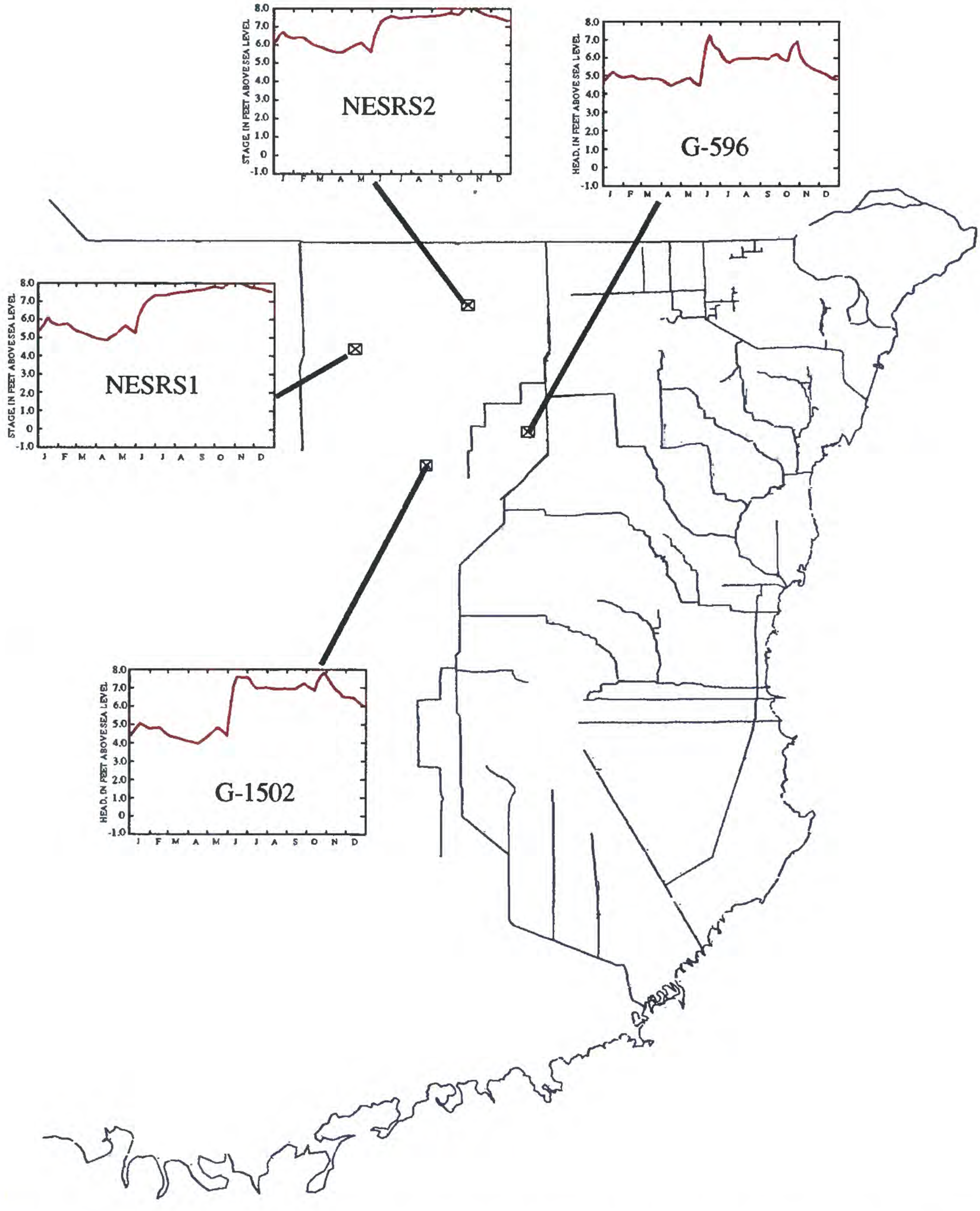

Figure 36. Simulated ground-water heads and wetlands stages (MODFLOW) produced by the RAFT model for flood scenario.

and a MODFLOW model were constructed for the southeastern wetlands in Dade County. Both models used the same grid and boundary conditions. Model results showed no conclusive advantage to either model in representing this particular situation. However, tidal fluctuations were not indicated at the field sites used for comparison.

A submodel of the southern Dade County model was constructed to predict the effects of remediation proposed in the Modified Water Deliveries Plan of the COE. The RAFT model was used to study the effects of higher water levels in the Everglades and proposed new canal and pumps to protect the 8.5-mi^2 residential area. Pumps in the model were given infinite capacity so their maximum capacity could be ascertained from model output. Maximum pumping rates of 400 and 150 ft^3/s were computed for the two proposed pumps in the area.

In conclusion, the field analysis of a coupled BRANCH/MODFLOW model indicates that the new model options are very useful in hydraulic modeling of areas with close connectivity between the surface water and ground water. These dynamic interactions require the use of dynamic models in order to properly represent flow in the system.

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APPENDIX

Effect of Grid Size on Leakage Coefficients

The leakage between the aquifer and river flow is computed with the same basic equation in all of the surface-water interaction packages described for MODFLOW, including MODBRANCH. This equation is derived from Darcy's law (McDonald and Harbaugh, 1988; Prudic, 1989; Swain and Wexler, 1993) and is:

$$q = \frac{K'}{b'} B (Z - h) \qquad \text{(A1)}$$

where:

q is leakage flow per unit length of channel,
K' is lateral hydraulic conductivity of riverbed,
b' is thickness of riverbed,
B is width of river,
Z is river stage, and
h is head in the aquifer.

In purely mathematical terms, b' is the effective distance between the point where the river stage is Z and the aquifer head is h.

The original formulation was made assuming a situation where a thin riverbed is much less permeable than the surrounding aquifer, so the aquifer head outside the riverbed is relatively spatially uniform at a value h. However, many situations are more properly visualized as in figure A1. In the case of a losing stream, the aquifer head slopes away from the canal gradually. In this case, b' is the distance from the canal to wherever the head in the aquifer is equal to h. When using equation A1 in a numerical model, h is the value of head in the aquifer cell containing the river. Given a numerical model that is functioning properly, the actual heads in the real aquifer area defined by the cell would have an average value equal to h. As shown in figure A1, the larger cell will have a different h than the small cell since it is representing an average head over a different area. This means that the head difference, $Z - h$, will be different for different cell sizes. Since there is only one correct value of q in equation A1, the effective distance b' should increase as the cell size increases, causing the same value of q to be computed always by equation A1.

In the computational model, K'/b' is a lumped parameter that is usually calibrated. It is not transferable between models of the same area that have different grid sizes. Since head gradients tend to be largest near the river, the leakage coefficient K'/b' is most sensitive to grid size for small grids. When the grid becomes very large relative to the spatial variations in head around the river, the leakage coefficient is relatively insensitive to grid size.

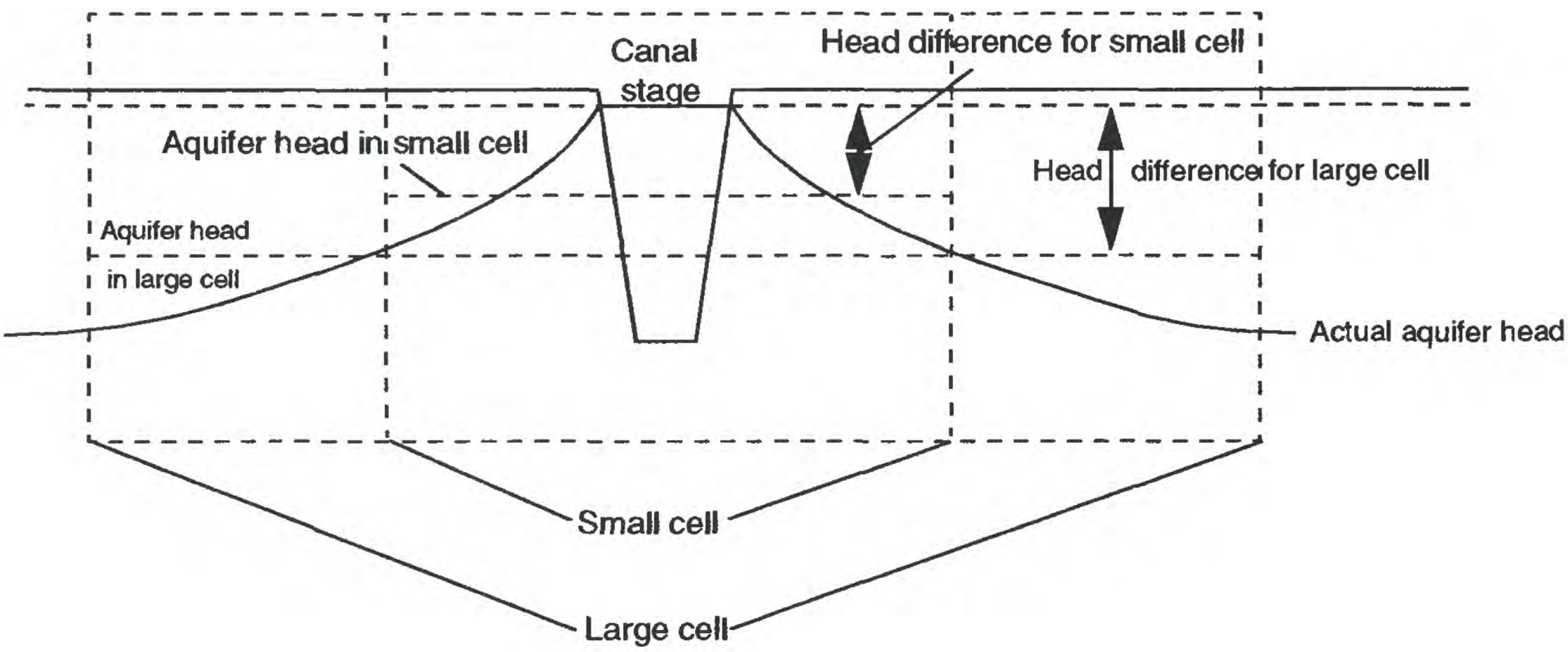

Figure A1. Relation of cell size to effective aquifer heads.