Evaluation of the Use of Reach Transmissivity to Quantify Leakage Beneath Levee 31N, Miami-Dade County, Florida

U.S. GEOLOGICAL SURVEY
Water-Resources Investigations Report 00-4066

Prepared as part of the
U.S. GEOLOGICAL SURVEY PLACE-BASED
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Evaluation of the Use of Reach Transmissivity to Quantify Leakage Beneath Levee 31N, Miami-Dade County, Florida

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Abstract

A coupled ground- and surface-water model (MODBRANCH) was developed to estimate ground-water flow beneath Levee 31N in Miami-Dade County, Florida, and to simulate hydrologic conditions in the surrounding area. The study included compilation of data from monitoring stations, measurement of vertical seepage rates in wetlands, and analysis of the hydrogeologic properties of the ground-water aquifer within the study area. In addition, the MODBRANCH code was modified to calculate the exchange between surface-water channels and ground water using a relation based on the concept of reach transmissivity.

The modified reach-transmissivity version of the MODBRANCH code was successfully tested on three simple problems with known analytical solutions. It was also tested and determined to function adequately on one field problem that had previously been solved using the unmodified version of the software. The modified version of MODBRANCH was judged to have performed satisfactorily, and it required about 60 percent as many iterations to reach a solution. Additionally, its input parameters are more physically-based and less dependent on model-grid spacing. A model of the Levee 31N area was developed and used with the original and modified versions of MODBRANCH, which produced similar output. The mean annual modeled ground-water heads differed by only 0.02 foot, and the mean annual canal discharge differed by less than 1.0 cubic foot per second.

Seepage meters were used to quantify vertical seepage rates in the Everglades wetlands area west of Levee 31N. A comparison between results from the seepage meters and from the computer model indicated substantial differences that seemed to be a result of local variations in the hydraulic properties in the topmost part of the Biscayne aquifer. The transmissivity of the Biscayne aquifer was estimated to be 1,400,000 square feet per day in the study area.

The computer model was employed to simulate seepage of ground water beneath Levee 31N. Modeled seepage rates were usually between 100 and 400 cubic feet per day per foot of levee, but extreme values ranged from about -200 to 500 cubic feet per day (positive values indicate eastward seepage beneath the levee). The modeled seepage results were used to develop an algorithm to estimate seepage based on head differential at selected monitoring stations. The algorithm was determined to adequately predict ground-water seepage.

INTRODUCTION

To manage water levels in the conservation areas and freshwater deliveries to Everglades National Park, it is important to determine the volume of water seeping from the water-conservation areas to the underlying aquifers. An accurate water budget to meet the competing natural and anthropogenic needs cannot be determined without this information. As part of the U.S. Geological Survey (USGS) place-based studies program, a study was conducted to evaluate methods for quantifying these seepage losses. The study site is located along Levee 31N and the adjacent canal in central Miami-Dade County (fig. 1). This levee is part of the eastern boundary of the Everglades

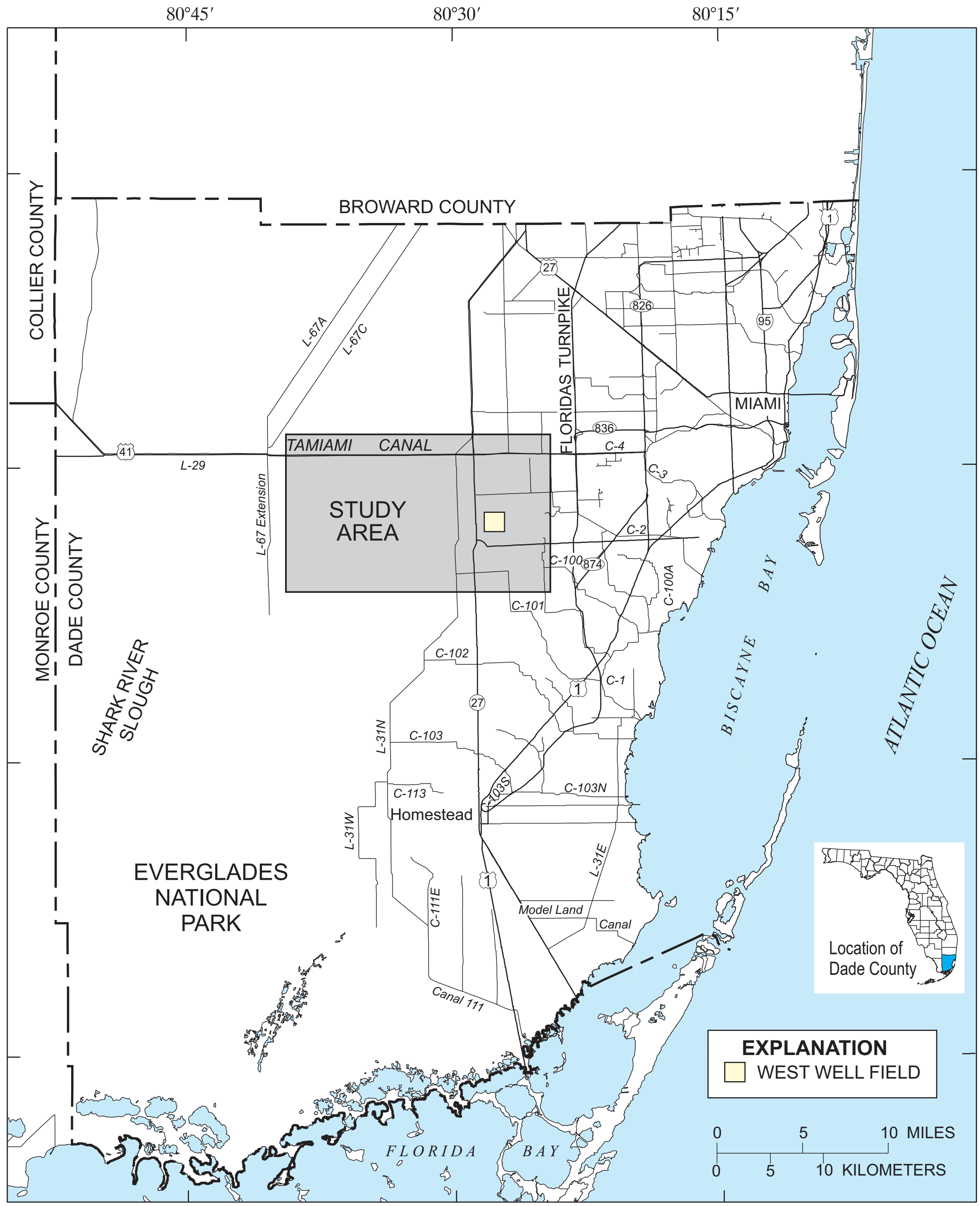

Figure 1. Location of study area, canals, and levees in Miami-Dade County.

wetlands. From the wetlands, water seeps into the Biscayne aquifer, which is about 70 ft (feet) thick in the area directly beneath Levee 31N; the aquifer thickens to the east. Due to high permeability of the aquifer, water flows relatively fast toward urban and agricultural areas to the east. Seepage to the aquifer from the Everglades is critical for maintaining water levels in water-supply wells to the east and for preventing the inland movement of saltwater from the coast. However, lowering ground-water levels to the east has resulted in increased ground-water flow eastward from Water Conservation areas 3A and 3B located northwest of Levee 31N, and reduced surface-water flows to the south. Levees 67A and 67C were constructed in Water Conservation Areas 3A and 3B to direct water southward toward the central region of Everglades National Park. This water-management scheme has been effective in delivering water to the southwest; however, it has reduced flow to the southeast (the northeastern part of Everglades National Park). Altering historic flow directions and water-level durations has adversely affected parts of the Everglades ecosystem.

Water managers are interested in restoring predevelopment flow conditions to the Everglades and are continually balancing the needs of maintaining (1) a healthy Everglades ecosystem, and (2) a sufficient municipal and agricultural water supply. Water in the northeastern part of Everglades National Park, located west of Levee 31N, as well as water in the Levee 31N Canal provide recharge for municipal well fields just east of Levee 31N. The canal also is used to deliver water to agricultural areas to the south. One of the needs of the restoration efforts is to account for all significant hydrologic inflows and outflows to the Everglades ecosystem. The seepage of ground water under Levee 31N constitutes a substantial outflow of water from this system. Restoration of the Everglades ecosystem will require a mosaic of accurate data to reveal a complete picture of this complex system.

Purpose and Scope

The purpose of this report is to describe the methods for quantifying ground-water seepage beneath Levee 31N, and to specify the data requirements and computational effort that these methods require. Modifications to the existing USGS model code MODBRANCH (Swain and Wexler, 1996) were made by adding the reach-transmissivity leakage option to more accurately represent the leakage conditions at the Levee 31N site. A finite-difference model was developed to analyze ground-water and surface-water flow in the vicinity of Levee 31N. A substantial amount of data was needed for model input and calibration. These data included geologic information, vertical seepage measurements, canal stage and discharge measurements, and ground-water levels. An algorithm suitable for application to real-time seepage estimation was developed. Accurate seepage data will enhance the accuracy of models of the Everglades and coastal systems. The methods evaluated in this report will be a critical element for water managers in their endeavors to restore historical flow patterns in the Everglades ecosystem.

Description of Study Area

The study area is limited to a 7-mi (mile) reach of Levee 31N in central Miami-Dade County (fig. 2). Levee 31N has a depth of about 20 ft and a top width of about 100 ft. The canal stage generally does not vary more than 3 ft during the year. The area is characterized by low topographic relief with elevations ranging from 4 to 8 ft above sea level. The Levee 31N study site is bordered by Tamiami Canal to the north, C-1W Canal to the south, Everglades National Park to the west, and agricultural and urban areas to the east (fig. 2).

Hydrogeology and Aquifer Characteristics

The hydrogeology and some aquifer characteristics of the study area are well defined based on previous studies by Causaras (1987) and Fish and Stewart (1991). The surficial aquifer system underlies central Miami-Dade County to a depth of about 180 ft below sea level (fig. 3). The unconfined Biscayne aquifer in the upper part of the surficial aquifer system consists of the Pamlico Sand, Miami Limestone, Anastasia Formation, Key Largo Limestone, and the Fort Thompson Formation all of Pleistocene age as well as contiguous, highly permeable beds of the Tamiami Formation of Pliocene and Miocene ages. The base of the Biscayne aquifer is present at the top of the upper clastic unit of the Tamiami Formation, and it extends from about 44 to 84 ft below sea level in the southwestern

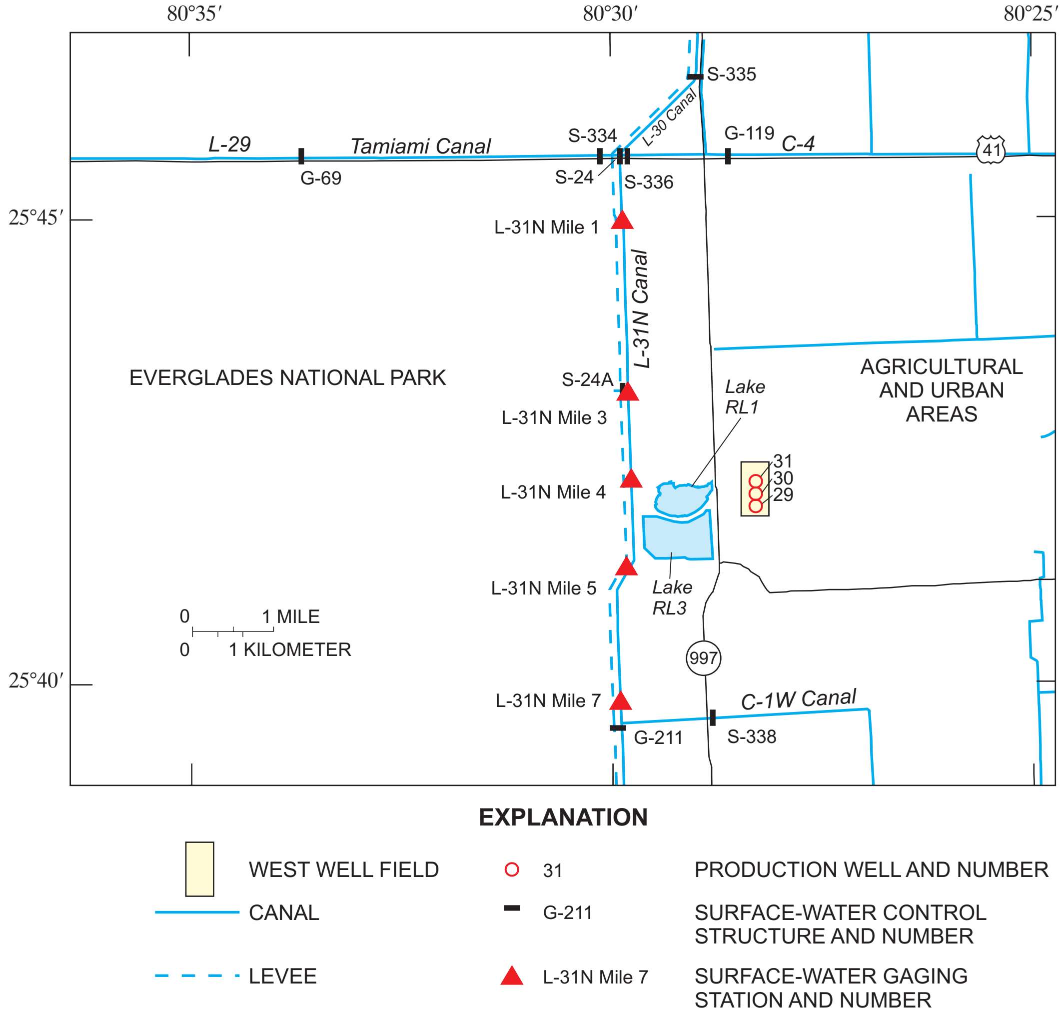

Figure 2. Study area showing primary canals, levees, control structures, and surface-water gaging stations.

and northeastern corners of the study area, respectively (fig. 3). Below the Biscayne aquifer are less permeable limestone, sand, and sandstone of the Tamiami Formation. A thin layer of organic material overlies the Biscayne aquifer in the Everglades wetlands area.

The hydraulic conductivity is estimated to be 29,000 ft/d (feet per day) in the Biscayne aquifer and 470 ft/d in the Tamiami Formation below the aquifer (Fish and Stewart, 1991, p. 28). There are two semiconfining layers of low-permeability limestone in the study area. The shallower semiconfining layer is about 2 ft thick and is located at the top of the Fort Thompson Formation, just below the Miami Limestone. The deeper semiconfining layer averages about 5 ft thick and has nearly the same slope as the upper surface of the Tamiami Formation (Causaras, 1987). Regional water-table maps indicate that ground water flows from west to east beneath Levee 31N (Fish and Stewart, 1991). However, operation of the West Well Field and the formation of two lakes, RL1 and RL3 (fig. 2), as a result of rock mining may affect flow patterns in the area. Water levels are usually higher in the western part of the study area (Everglades) than in the eastern part (agricultural and urban areas).

Rainfall and Levees

The Biscayne aquifer is recharged by rainfall in upland areas that infiltrates directly to the aquifer or by surface water that seeps downward through wetland sediments to the aquifer. The subtropical climate of southeastern

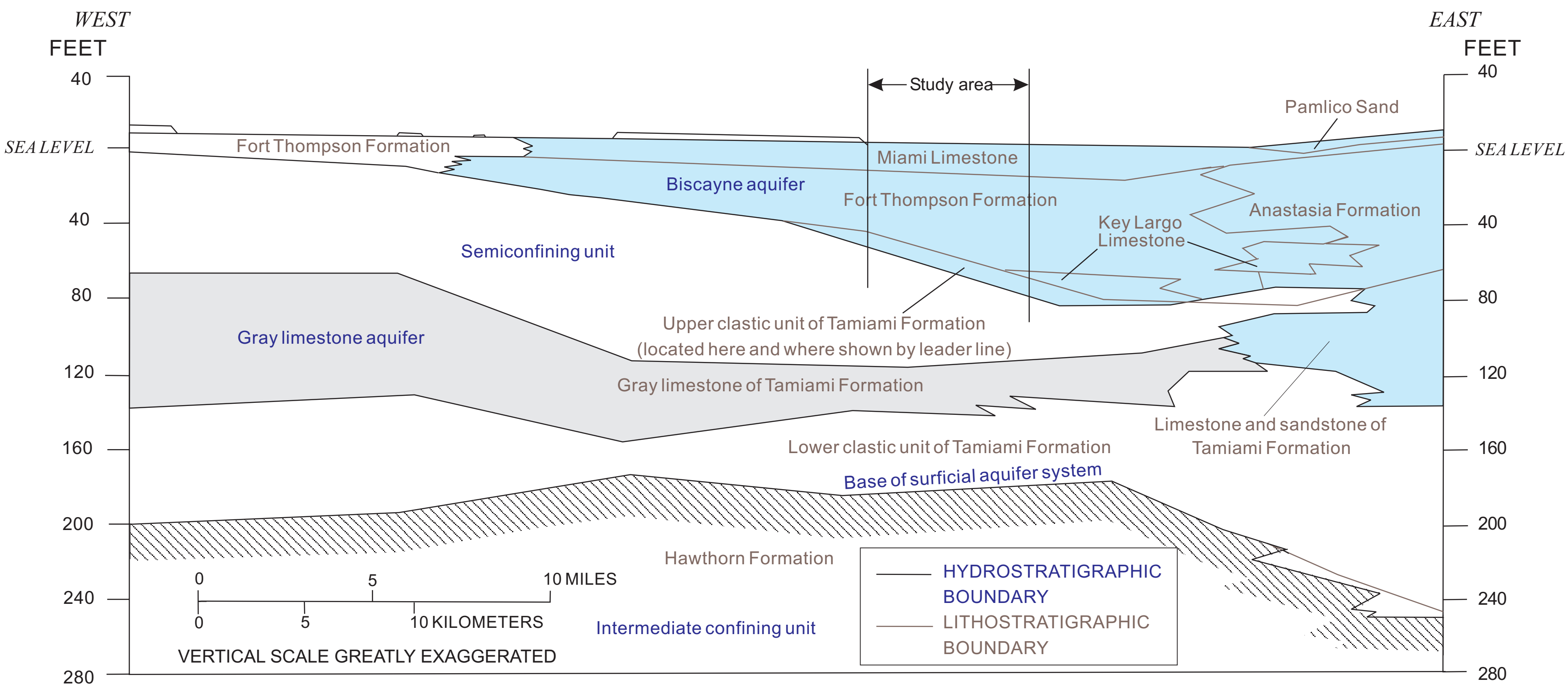

Figure 3. Hydrogeologic section showing formations, aquifers, and confining units of the surficial aquifer system in central Miami-Dade County, (modified from Fish and Stewart, 1991). Section extends along Tamiami Canal.

Florida consists of hot, wet summers and mild, dry winters. About 70 percent of the total annual rainfall occurs from June to October (Jordan, 1984, p. 22).

In 1953, Levees 31N, 30 (to the north) and 31W (south of the study area) were constructed to store excess water during the wet season and transfer the excess water to areas of need during the dry season (Fish and Stewart, 1991, p. 8). This system of levees allows flooding in the Everglades and reduces the probability of flooding east of the levees in central Miami-Dade County. There are several control structures on the canals that are used to regulate the flow or prevent saltwater intrusion. The operation of these structures was not used in modeling efforts for this study; recorded data from some of the structures were used to determine boundary conditions.

Ground- and Surface-Water Numerical Modeling

Most computer models of ground-water flow are based on Darcy’s law, whereas most surface-water models employ the de Saint Venant equations. Additionally, surface-water systems tend to respond more quickly to changing boundary conditions than ground-water systems; the time scale of interest is usually substantially longer for analysis of ground water than for analysis of surface water. As a result, it is difficult to develop a single model that handles both ground-water and surface-water flow at the highest level of detail.

Despite these difficulties, considerable efforts have been made to develop a few models capable of detailed analysis of both ground water and surface water. An example of such a model is MODBRANCH (Swain and Wexler, 1996), which links MODFLOW (McDonald and Harbaugh, 1988), a three-dimensional finite-difference ground-water model, to BRANCH (Schaffranek and others, 1981), a one-dimensional model of unsteady flow in open channel networks. The MODBRANCH code is one of several modules of MODFLOW; MODBRANCH links BRANCH to MODFLOW by means of a calculation of leakage through the bottom of the surface-water channels. The MODBRANCH name will be used herein to refer to the combined MODFLOW/BRANCH ground-water/surface-water model.

The current leakage calculation of MODBRANCH employs a vertical-flow relation, which assumes that all leakage occurs through a low-permeability layer on the bottom of the surface-water channel. An alternate method of calculating leakage, the reach-transmissivity relation (Morel-Seytoux, 1975; Morel-Seytoux and Daly, 1975), accounts for leakage throughout the wetted perimeter of the channel. A number of differences exist between the vertical-flow and reach-transmissivity relations in addition to the portion of the channel cross-section through which leakage occurs. The reach-transmissivity relation was incorporated into a modified version of MODBRANCH; this required modification of the source code. A comparison of the two relations was made to evaluate the performance of the modification.

Previous Studies

Several hydrologic, modeling, and seepage studies have been conducted to evaluate flow beneath levees and between canals and the Biscayne aquifer in southeastern Florida. Seepage beneath a test levee prior to construction of Levee 30 was evaluated by the U.S. Army Corps of Engineers (1952). Seepage beneath Levee 30 at its northern end was evaluated by Klein and Sherwood (1961). Seepage from Lake Okeechobee was evaluated by Meyer and Hull (1968) and McKenzie (1973). The effect of canal bottom sediments on infiltration from Miami Canal into the Biscayne aquifer was evaluated by Miller (1978). Ground-water flow beneath Levee 35A in Broward County was evaluated by Swayze (1988). Chin (1990) used the reach-transmissivity leakage relation to evaluate leakage in the Levee 31N Canal under steady-state conditions. Swain and others (1996) developed a model of ground water and surface water for a large region of the Everglades that included Levee 31N Canal. Use of seepage meters to quantify vertical exchange in wetlands was investigated by Harvey (1996). R.S. Sonenshein (U.S. Geological Survey, written commun., 2000) evaluated methods for quantifying ground-water seepage beneath Levee 30.

Acknowledgments

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MODIFICATIONS TO MODBRANCH LEAKAGE CALCULATIONS

The MODBRANCH software links the MODFLOW ground-water model with the BRANCH surface-water model by calculating the exchange of water between the ground-water aquifer and the surface-water channel. The existing version of MODBRANCH (Swain and Wexler, 1996) assumes that leakage occurs through a low-permeability layer at the bottom of the surface-water channel. A modified version of MODBRANCH, with calculations based on a different conceptualization of leakage, was developed and tested.

Formulation of Leakage Relations

The leakage relation currently employed by MODBRANCH to quantify exchange between the ground-water and surface-water systems is designated as the "vertical-flow" relation; it assumes vertical flow through a low-permeability layer at the bottom of the channel. The mathematical formulation of this relation is based on Darcy's law and may be expressed as follows:

$$q = \frac{K'}{b'}B(Z - h), \qquad (1)$$

where

q is leakage to the aquifer from the channel (volume of water per unit channel length per unit time),
K' is hydraulic conductivity of the low-permeability layer of the channel bottom,
b' is thickness of the low-permeability layer of the channel bottom,
B is top width of the channel,
Z is surface-water elevation in the channel, and
h is elevation of the water table directly adjacent to the channel.

The vertical-flow relation assumes that the sides of the channel are impermeable; all leakage occurs through the bottom of the channel, which results in a discontinuity in water-surface elevation between the channel and aquifer at the lateral boundary of the channel. A schematic representation of the vertical-flow leakage relation is shown in figure 4.

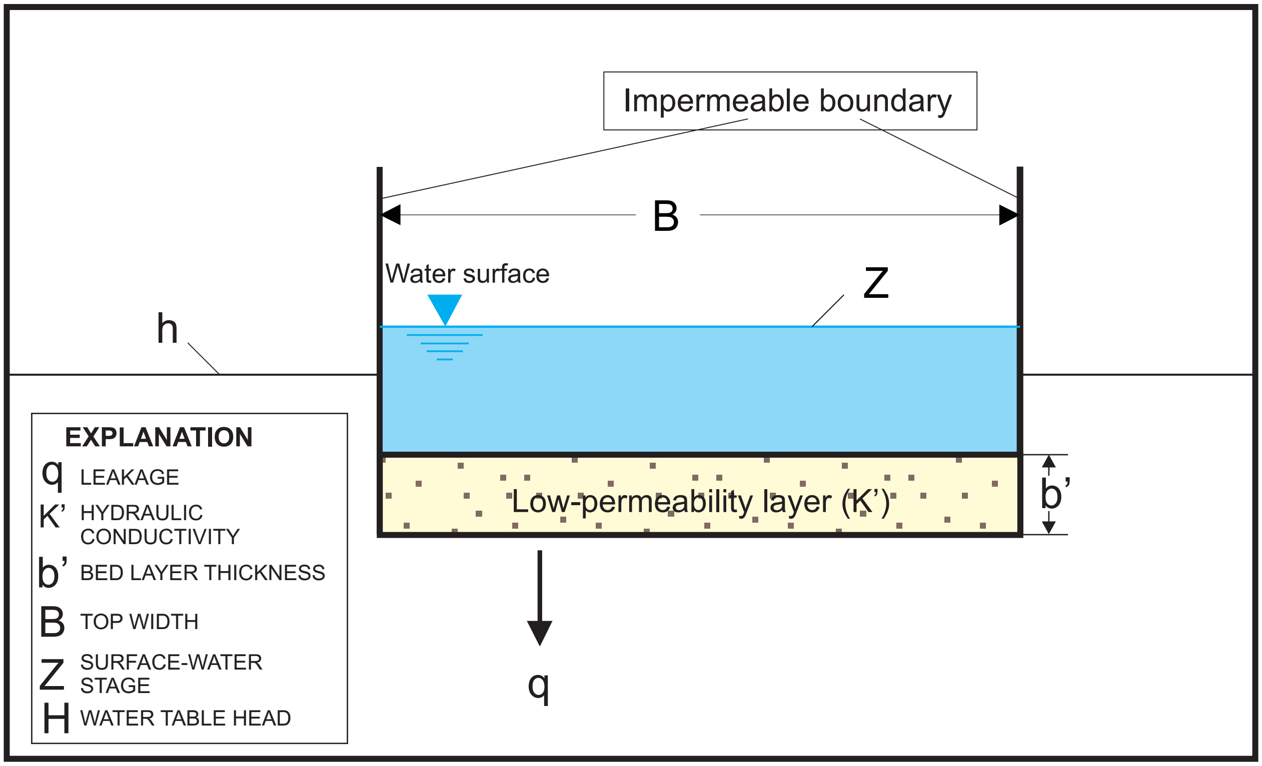


Figure 4. Vertical-flow leakage relation.

An alternate leakage relation can be developed using the concept of reach transmissivity (Morel-Seytoux, 1975; Morel-Seytoux and Daly, 1975; Illangasekare and Morel-Seytoux, 1986). This relation was selected for this study because of its simplicity and the availability of independent physical measures of the coefficients; additionally, the reach-transmissivity relation responds differently to local changes in surface-water elevations than the vertical-flow relation does. Furthermore, the reach-transmissivity relation functions well under steady-state conditions at the L-31N Canal (Chin, 1991), and is suitable for analysis of transient conditions (Mishra and Seth, 1988; Nemeth, 2000). In general form, the reach-transmissivity relation is expressed as:

$$q = \Gamma_r (Z - h) \quad , \tag{2}$$

where Γ_r is the reach transmissivity or volumetric flow rate of water per unit drawdown per unit channel length, and h is the ground-water head measured at a specified distance, L, away from the channel. The parameter Γ_r is dependent on the geometry of the channel cross section and the characteristics of the channel bed. In certain cases, however, it is possible to make simplifying assumptions. Unless the channel width is small relative to the thickness of the aquifer, the channel can be assumed to be fully penetrating even if it is not. This assumption can be made at the L-31N Canal where the top width of the canal is about 1.3 times as large as the aquifer thickness. In such a case, the Dupuit-Forcheimer assumption can be used to help derive an estimate of reach transmissivity. The derivation begins with Darcy's law, written for one-dimensional flow per unit width:

$$q = -Kh\frac{dz}{dx}\,, \tag{3}$$

where K is the hydraulic conductivity of the aquifer, h is the saturated thickness of the aquifer, and dz/dx is the horizontal head gradient. In the case of a surface-water channel, leakage occurs to both sides. Under symmetrical conditions (the ground-water head is the same on both sides of the channel), the following equation is valid:

$$q = -2Kh\frac{dz}{dx}\,. \tag{4}$$

The aquifer transmissivity, T, is the product of the hydraulic conductivity and thickness. The finite-difference form of the reach-transmissivity equation is as follows:

$$q = \frac{2T(Z-h)}{L-\frac{B}{2}}\,. \tag{5}$$

In this formulation, the reach-transmissivity coefficient is, therefore, expressed as follows:

$$\Gamma_r = \frac{2T}{L-\frac{B}{2}}\,. \tag{6}$$

Schematic representations of the reach-transmissivity relation for symmetrical and asymmetrical conditions are shown in figure 5. The reach-transmissivity equation can be made to account for asymmetrical conditions; leakage to each side of the channel can be expressed as:

$$q_i = \frac{T(Z-h_i)}{L_i-\frac{B}{2}}\,, \tag{7}$$

where subscript i designates the side of the channel to which leakage occurs. The total leakage from the channel to the aquifer can be written as:

$$q = \left[\frac{T_R(Z-h_R)}{L_R-\frac{B}{2}} + \frac{T_L(Z-h_L)}{L_L-\frac{B}{2}}\right]\,, \tag{8}$$

where the subscripts R and L indicate the right and left sides of the channel, respectively.

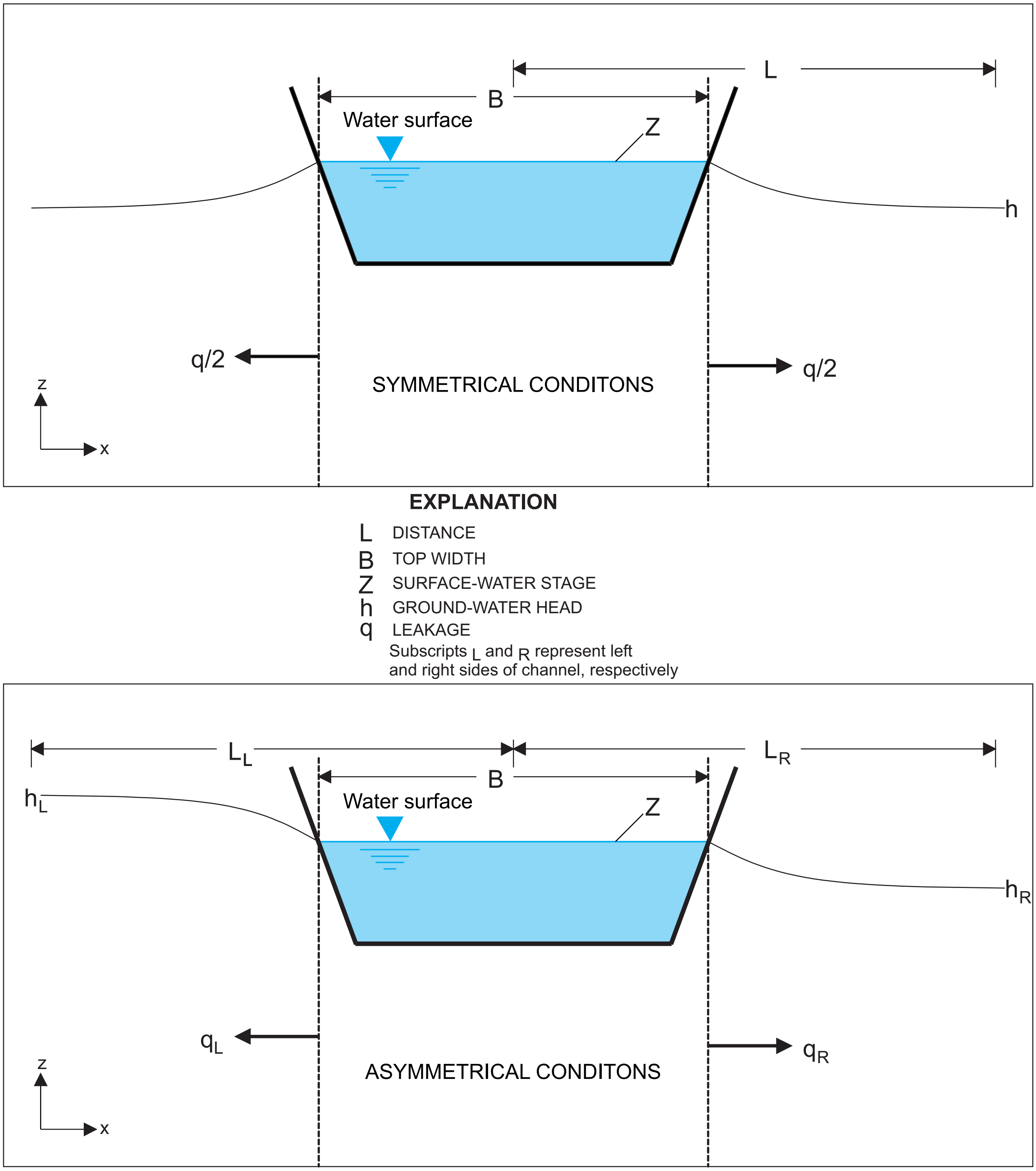


Figure 5. Reach-transmissivity leakage relation for symmetrical and asymmetrical conditions.

Functional Differences Between Leakage Relations

The vertical-flow and reach-transmissivity leakage relations are based on differing conceptualizations of leakage. The vertical-flow relation assumes that the sides of the channel are impermeable and that all leakage occurs through the bottom. In contrast, leakage in the reach-transmissivity relation occurs through the sides of the channel, and separate leakage components are computed for the left and right sides of the channel. The water-table elevation directly adjacent to the channel is measured to determine the head differential between the channel and ground water in the vertical-flow relation. In the reach-transmissivity relation, the water-table elevation is measured a substantial distance away from the channel (although the exchange of water between the ground- and surface-water models still occurs in the model cell in which the channel is located). This results in a discontinuity in water-surface elevation at the channel boundary in the vertical-flow conception, whereas the water surface is continuous at the channel boundary in the reach-transmissivity relation. Another difference between the leakage relations is that to make coefficient determination practical, the reach-transmissivity relation requires that the channel be assumed to fully penetrate the aquifer; this assumption is not necessary for use of the vertical-flow relation.

In most cases, the stage in surface-water channels fluctuates on a much shorter time scale than levels in the water table. As a result, the most dramatic fluctuations of the water table usually occur directly adjacent to the channels. The vertical-flow and reach-transmissivity leakage relations may provide different results when modeling short-term canal fluctuations. When used in a finite-difference model, the vertical-flow relation employs a reference aquifer head within the same model cell as the channel; the head in this cell is the first to respond to changes in the canal stage and fluctuates more than the head of cells at a greater distance from the canal. The reach-transmissivity relation, however, uses a reference aquifer head in a cell more distant from the channel, and consequently, is less affected by fluctuations in channel stage. This particular difference between the two leakage relations has the potential to result in differences in the quantity and temporal distribution of exchange between ground water and surface water.

Parameters in the reach-transmissivity relation have a different physical basis than those of the vertical-flow relation. The reach-transmissivity coefficient, Γ_r, is calculated from the channel top width, the distance from the channel to the point of aquifer head measurement, and aquifer transmissivity; the first two parameters are distances that are easy to obtain and calculate. The transmissivity of the aquifer is generally available from published sources or pump tests, particularly in areas where previous ground-water research has been conducted.

In contrast, the vertical-flow relation requires determination of the hydraulic conductivity and thickness of a low-permeability layer at the channel bed. These data are not as readily obtainable and usually cannot be known without collecting samples from the channel bed. Furthermore, determination of these parameters becomes even more problematic if the leakage does not actually conform to the vertical-flow conceptualization (that is, all the leakage does not occur through a clearly definable low-permeability layer in the bottom of the channel). This condition commonly occurs in unlined artificial channels that have been excavated from the aquifer material, resulting in the absence of a low-permeability bed layer. Additionally, the vertical-flow relation may not be an accurate conceptualization of flow in natural channels where the surface-water stage is much higher than the elevation of the water table. In this case, the water exiting the channel must flow a great distance through the aquifer material. The head loss associated with flow through the bed layer is usually small in comparison to head losses resulting from flow through a greater distance of aquifer material.

When the vertical-flow relation does not represent the actual leakage mechanisms, the channel bed thickness and hydraulic conductivity become calibration parameters that are developed by trial and error rather than by direct measurement. As a result, the bed thickness, b', becomes defined mathematically as the distance from the point in the channel where the stage is Z to the point in the model cell where the head is h (Swain and others, 1996). A numerical model employing the vertical-flow relation requires input of the lumped parameter K'/b'. If a low-permeability layer at the channel bed does not exist, leakage occurs directly into the aquifer material, so K' becomes the hydraulic conductivity of the entire aquifer rather than a discrete bed layer. By definition, b' must be smaller than the largest dimension of each model cell; it is apparent that b' must decrease as the cell size decreases. However, the aquifer's hydraulic conductivity, K', is essentially independent of cell size. Therefore, K'/b' increases as the cell size decreases, meaning that the calibrated leakage parameter in the vertical-flow relation is dependent on the model-grid spacing.

This phenomenon also can be explained by considering the relation between cell size and ground-water heads. The ground-water head is equal to the channel stage at the bank of the channel and slopes continuously toward the ambient head; the influence of the channel on the ground-water head decreases as distance from the channel increases. If the model cells are very large, then the average ground-water head in each cell is only slightly affected by the channel stage. However, if the cell spacing is small, then the presence of the channel substantially affects the ground-water head in cells near the channel; the average head, *h*, in the cell containing the channel becomes dependent on the size of the model grid. Obviously, this situation is not desirable because it means that leakage parameters calibrated for one grid spacing cannot be applied to any other grid spacing. The reach-transmissivity relation is not subject to this problem. The reach-transmissivity equation is only applicable when the Dupuit-Forcheimer assumption is valid; the distance, *L*, must be sufficiently large that the ground-water head used as an input to the equation is not substantially affected by vertical gradients caused by the presence of the channel. As long as this condition is maintained, calibration of input parameters of the reach-transmissivity relation is not dependent on model grid spacing. The distance, *L*, must conform to the grid spacing, but a transmissivity value calibrated at one grid spacing will remain valid if the grid is changed.

The reach-transmissivity relation offers important benefits over the vertical-flow leakage relation. The input parameters required to use the reach-transmissivity relation are more easily obtained and, in many cases, have a more legitimate physical basis. As a result, the amount of model calibration by trial and error is substantially reduced. Additionally, the input parameters of the vertical-flow relation are dependent on the model grid spacing. This dependence is particularly important when leakage does not actually conform to the vertical-flow conceptualization. In the reach-transmissivity relation, only the distance to the ground-water head reference cell is dependent on grid spacing; the other input parameters are not affected.

Finite-Difference Form of Leakage Relations

Currently, MODBRANCH calculates leakage using the vertical-flow relation. The incorporation of the reach-transmissivity relation into MODBRANCH requires modification of the leakage equations within the FORTRAN code. In finite-difference form, the continuity equation for surface-water flow is as follows (Swain and Wexler, 1996):

$$\begin{aligned}\bar{B}\left[\frac{Z_{i+1}^{j+1}+Z_i^{j+1}}{2\Delta t}-\frac{Z_{i+1}^{j}+Z_i^{j}}{2\Delta t}\right]+\theta\frac{Q_{i+1}^{j+1}-Q_i^{j+1}}{\Delta X_i}+(1-\theta)\frac{Q_{i+1}^{j}+Q_i^{j}}{\Delta X_i}\\+\frac{\chi}{2}\left[C_{i+1}B_{i+1}^{j+1}(Z_{i+1}^{j+1}-h^{j+1})+C_iB_i^{j+1}(Z_i^{j+1}-h^{j+1})\right]\\+\frac{(1-\chi)}{2}\left[C_{i+1}B_{i+1}^{j}(Z_{i+1}^{j}-h^{j})+C_iB_i^{j}(Z_i^{j}-h^{j})\right]=0\end{aligned}, \tag{9}$$

where

$\bar{B}$ is average channel top width from the previous time step,

Z is channel stage,

Δt is time-step length,

θ is weighting factor for spatial derivatives,

Q is flow rate of the channel,

ΔX_i is length of the channel segment from points *i* to *i+1*,

χ is weighting factor for averaged quantities,

C is the MODBRANCH leakage coefficient (k'/b'),

B is channel top width, and

h is ground-water head.

The subscripts and superscripts refer to space and time, respectively. The quantity $\bar{B}$ is defined as:

$$\bar{B} = \chi\frac{B_{i+1}^{j} + B_{i}^{j}}{2} + (1-\chi)\frac{B_{i+1}^{j-1} + B_{i}^{j-1}}{2} \quad . \tag{10}$$

Additionally, in the continuity equation, C is K'/b' from the vertical-flow leakage relation. The quantity K'/b' is an input parameter to the MODBRANCH package; replacement of the vertical-flow relation with the reach-transmissivity relation requires introduction of a new input parameter in place of K'/b'.

The continuity equation, in finite-difference form with the inclusion of leakage terms, is solved simultaneously with the finite-difference form of the momentum equation in BRANCH (Schaffranek and others, 1981). These equations are solved through the use of a matrix solution that requires the continuity equation to be transformed into the following form (Swain and Wexler, 1996):

$$Q_{i+1}^{j+1} + \gamma Z_{i+1}^{j+1} - Q_{i}^{j+1} + \alpha Z_{i}^{j+1} = \delta \, , \tag{11}$$

where

$$\gamma = \frac{\bar{B}\Delta X_i}{2\Delta t\theta} + \frac{\chi C_{i+1} B_{i+1}^{j+1}\Delta X_i}{2\theta} \, , \tag{12}$$

$$\alpha = \frac{\bar{B}\Delta X_i}{2\Delta t\theta} + \frac{\chi C_i B_i^{j+1}\Delta X_i}{2\theta} \, , \tag{13}$$

and

$$\begin{aligned}\delta = {} & -\frac{1-\theta}{\theta}(Q_{i+1}^{j} - Q_{i}^{j}) + \left[\frac{\bar{B}\Delta X_i}{2\Delta t\theta} - (1-\chi)(C_{i+1}B_{i+1}^{j})\frac{\Delta X_i}{2\theta}\right]Z_{i+1}^{j} + \left[\frac{\bar{B}\Delta X_i}{2\Delta t\theta} - (1-\chi)(C_i B_i^{j})\frac{\Delta X_i}{2\theta}\right]Z_i^{j} \\ & + \frac{\Delta X_i}{2\theta}\left[\chi(C_{i+1}B_{i+1}^{j+1}h^{j+1} + C_i B_i^{j+1}h^{j+1}) + (1-\chi)(C_{i+1}B_{i+1}^{j}h^{j} + C_i B_i^{j}h^{j})\right]\end{aligned} \quad . \tag{14}$$

When the reach-transmissivity relation is employed, the finite form of the continuity equation for surface-water flow is:

$$\bar{B}\left[\frac{Z_{i+1}^{j+1}+Z_{i}^{j+1}}{2\Delta t}-\frac{Z_{i+1}^{j}+Z_{i}^{j}}{2\Delta t}\right]+\theta\frac{Q_{i+1}^{j+1}-Q_{i}^{j+1}}{\Delta X_i}+(1-\theta)\frac{Q_{i+1}^{j}-Q_{i}^{j}}{\Delta X_i}$$
$$+\frac{\chi}{2}\left[\left(\frac{T}{L_R-\frac{B}{2}}\right)_{i+1}(Z_{i+1}^{j+1}-h_R^{j+1})+\left(\frac{T}{L_R-\frac{B}{2}}\right)_{i}(Z_{i}^{j+1}-h_R^{j+1})\right]$$
$$+\frac{(1-\chi)}{2}\left[\left(\frac{T}{L_R-\frac{B}{2}}\right)_{i+1}(Z_{i+1}^{j}-h_R^{j})+\left(\frac{T}{L_R-\frac{B}{2}}\right)_{i}(Z_{i}^{j}-h_R^{j})\right] \quad , \qquad (15)$$
$$+\frac{\chi}{2}\left[\left(\frac{T}{L_L-\frac{B}{2}}\right)_{i+1}(Z_{i+1}^{j+1}-h_L^{j+1})+\left(\frac{T}{L_L-\frac{B}{2}}\right)_{i}(Z_{i}^{j+1}-h_L^{j+1})\right]$$
$$+\frac{(1-\chi)}{2}\left[\left(\frac{T}{L_L-\frac{B}{2}}\right)_{i+1}(Z_{i+1}^{j}-h_L^{j})+\left(\frac{T}{L_L-\frac{B}{2}}\right)_{i}(Z_{i}^{j}-h_L^{j})\right] = 0$$

When the reach-transmissivity relation is employed, the finite-difference form of the continuity equation is solved with the same type of matrix solution used for the vertical-flow relation. The expressions for the coefficients of continuity (eq. 10) must be transformed into the following set of equations:

$$\gamma = \frac{\bar{B}\Delta X_i}{2\Delta t\theta}+\frac{\chi\Delta X_i}{2\theta}\left[\left(\frac{T}{L_R-\frac{B}{2}}\right)_{i+1}+\left(\frac{T}{L_L-\frac{B}{2}}\right)_{i+1}\right] , \qquad (16)$$

$$\alpha = \frac{\bar{B}\Delta X_i}{2\Delta t\theta}+\frac{\chi\Delta X_i}{2\theta}\left[\left(\frac{T}{L_R-\frac{B}{2}}\right)_{i}+\left(\frac{T}{L_L-\frac{B}{2}}\right)_{i}\right] , \qquad (17)$$

and

$$\delta = \frac{1-\theta}{\theta}(Q_{i+1}^{j} - Q_{i}^{j}) + \left[\frac{\bar{B}\Delta X_i}{2\Delta t\theta} - (1-\chi)\frac{\Delta X_i}{2\theta}\left\{\left(\frac{T}{L_R - \frac{B}{2}}\right)_{i+1} + \left(\frac{T}{L_L - \frac{B}{2}}\right)_{i+1}\right\}\right] Z_{i+1}^{j}$$

$$+ \left[\frac{\bar{B}\Delta X_i}{2\Delta t\theta} - (1-\chi)\frac{\Delta X_i}{2\theta}\left\{\left(\frac{T}{L_R - \frac{B}{2}}\right)_{i} + \left(\frac{T}{L_L - \frac{B}{2}}\right)_{i}\right\}\right] Z_{i}^{j} \qquad (18)$$

$$+ \frac{\Delta X_i}{2\theta}\left[\begin{array}{l} \chi\left\{\left(\frac{T}{L_R - \frac{B}{2}}\right)_{i+1} h_R{}^{j+1} + \left(\frac{T}{L_L - \frac{B}{2}}\right)_{i+1} h_L{}^{j+1} + \left(\frac{T}{L_R - \frac{B}{2}}\right)_{i} h_R{}^{j+1} + \left(\frac{T}{L_L - \frac{B}{2}}\right)_{i} h_L{}^{j+1}\right\} \\ + (1-\chi)\left\{\left(\frac{T}{L_R - \frac{B}{2}}\right)_{i+1} h_R{}^{j} + \left(\frac{T}{L_L - \frac{B}{2}}\right)_{i+1} h_L{}^{j} + \left(\frac{T}{L_R - \frac{B}{2}}\right)_{i} h_R{}^{j} + \left(\frac{T}{L_L - \frac{B}{2}}\right)_{i} h_L{}^{j}\right\} \end{array}\right].$$

Examination of the equations of the two leakage relations reveals that the quantity $\frac{K'}{b'}B$ (also designated as *CB*) in the vertical-flow relation is equivalent to the quantity, $[T/(L_R - B/2) + T/(L_L - B/2)]$ in the reach-transmissivity finite-difference equation. However, *B* is not part of the leakage coefficient when the vertical-flow relation is employed; the coefficient is $\frac{K'}{b'}B$ (or *C*). As a result, $[T/(L_R - B/2)$ and $T/(L_L - B/2)]$ replace K'/b' in the MODBRANCH input, even though they replace $\frac{K'}{b'}B$ in the application of the finite-difference equation.

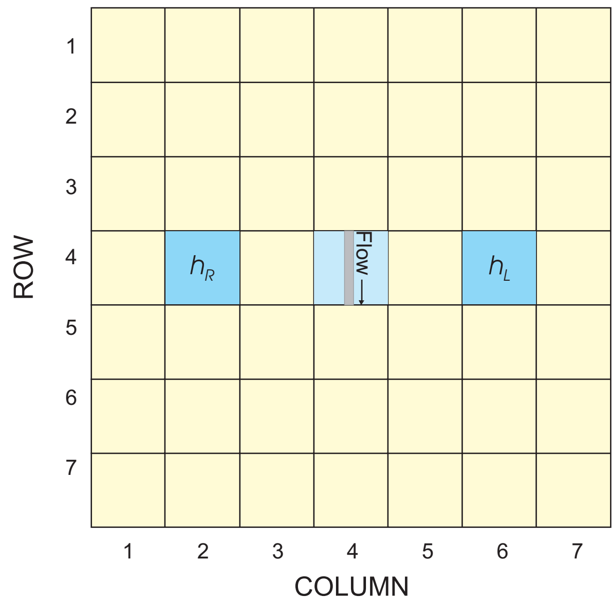
In MODBRANCH, *B* is a variable that is continually recalculated as conditions change. The quantities L_R-*B/2* and L_L-*B/2*, required as part of the input to the modified MODBRANCH, are specified as constants throughout each model run and should reflect the average channel width during the simulation period. Caution should be exercised when using the reach-transmissivity version of MODBRANCH when it is expected that variations in the channel width will be substantial relative to the total distance from the center of the channel to the point at which ground-water head is obtained.

Another required modification is that the ground-water reference head, *h*, must be separated into two quantities, h_R and h_L, which designate the ground-water head on the right and left sides of the channel, respectively. Obviously, the quantities required for input to the reach-transmissivity relation are considerably more complex in formulation than K'/b'. However, all of the parameters contributing to them may be readily obtained, in contrast with K'/b', which commonly is a calibration parameter without substantial physical basis. Additionally, leakage to the left and right sides of the channel are calculated separately and then summed when the reach-transmissivity relation is employed. As a result, calculated leakage to each side of the channel can be reported in the program's output in addition to the net leakage; this is not possible when the vertical-flow relation is used.

Another difference between the vertical-flow and reach-transmissivity leakage relations is the location where the ground-water head (*h* in the finite-difference form of the continuity equation) is measured. The vertical-flow relation uses the ground-water head in the same model cell where the channel is located, whereas the reach-transmissivity relation employs the ground-water head one or more cells distant from each side of the channel. To incorporate the reach-transmissivity relation, the MODBRANCH input was altered to include specification of the model cells from which the ground-water head, *h*, could be used for leakage computations; one cell on each side of the channel must be specified. However, leakage to or from the channel still occurs in the aquifer cell in which the channel is located (this is the same as for the vertical-flow relation). A more detailed discussion of the specific modifications made to the program code, including additional input requirements, is provided in the appendix.

Model Results and Analytical Solutions

To ensure that the finite-difference form of the reach-transmissivity equation was formulated correctly and solved properly in the modified version of MODBRANCH, tests were conducted on three simple theoretical problems with known analytical solutions. A one-layer, seven-row, seven-column grid was used in all three problems (fig. 6); row and column spacing was 100 ft. The surface-water channel was located in the center cell (row 4, column 4); and was defined to be a prismatic, rectangular channel with a width of 10 ft. The channel invert was at an elevation of 0 ft; stage was set to a constant value of 5 ft throughout all of the test simulations. The input values of the leakage coefficients, $T/(L_R - B/2)$ and $T/(L_L - B/2)$, were both specified to be 0.0005 ft/s (foot per second). The reference ground-water heads on the right and left sides of the channel (flow was assumed to be from top to bottom) were obtained from the model cells in row 4, columns 2 and 6, respectively. Values of the individual components of the leakage coefficients were as follows: B = 10 ft, L_R = 200 ft, L_L = 200 ft, and T = 0.0975 ft^2/s (square foot per second). The length of each simulation was 1 hour. Positive leakage values indicate flow of water from the channel to the aquifer; in addition to reporting the net leakage, the model output also included the leakage to the right and left sides of the channel.



h_R and h_L indicate the locations of the right and left ground-water head reference cells, respectively

Figure 6. Model grid for theoretical problems.

Theoretical problem 1 consisted of a constant stage channel in an aquifer with a constant head of 4 ft. Thus, the channel stage was 1 ft above the aquifer head (fig. 7). The analytical solution was obtained using equation 7; leakage per foot of channel length was calculated to be 0.001 ft^2/s. Multiplying this value by 100 ft of channel and 1 hour yielded a total leakage of 360 ft^3(cubic feet), which was correctly calculated by the model (180 ft^3 flowed into the channel from each side).

In theoretical problem 2, the ground-water head was specified to be 1 ft above the channel stage on the right side of the channel and 1 ft below the channel stage on the left side (fig. 7). Therefore, leakage into the channel was equal to leakage out of the channel, and the net leakage, as calculated by equation 8, was zero. The model results again corresponded to the analytical solution; 180 ft^3 flowed into the channel from the right side and 180 ft^3 flowed out of the channel from the left.

In theoretical problem 3, the ground-water head was 2 ft above the channel stage on the right side and 1 ft below the channel stage on the left side (fig. 7). Using equation 7 resulted in a leakage of 0.0005 ft^2/s per foot of channel length and a total leakage of -180 ft^3 over the period of simulation. The model results also yielded a total leakage of -180 ft^3 with -360 ft^3 on the right side of the channel and 180 ft^3 on the left side.

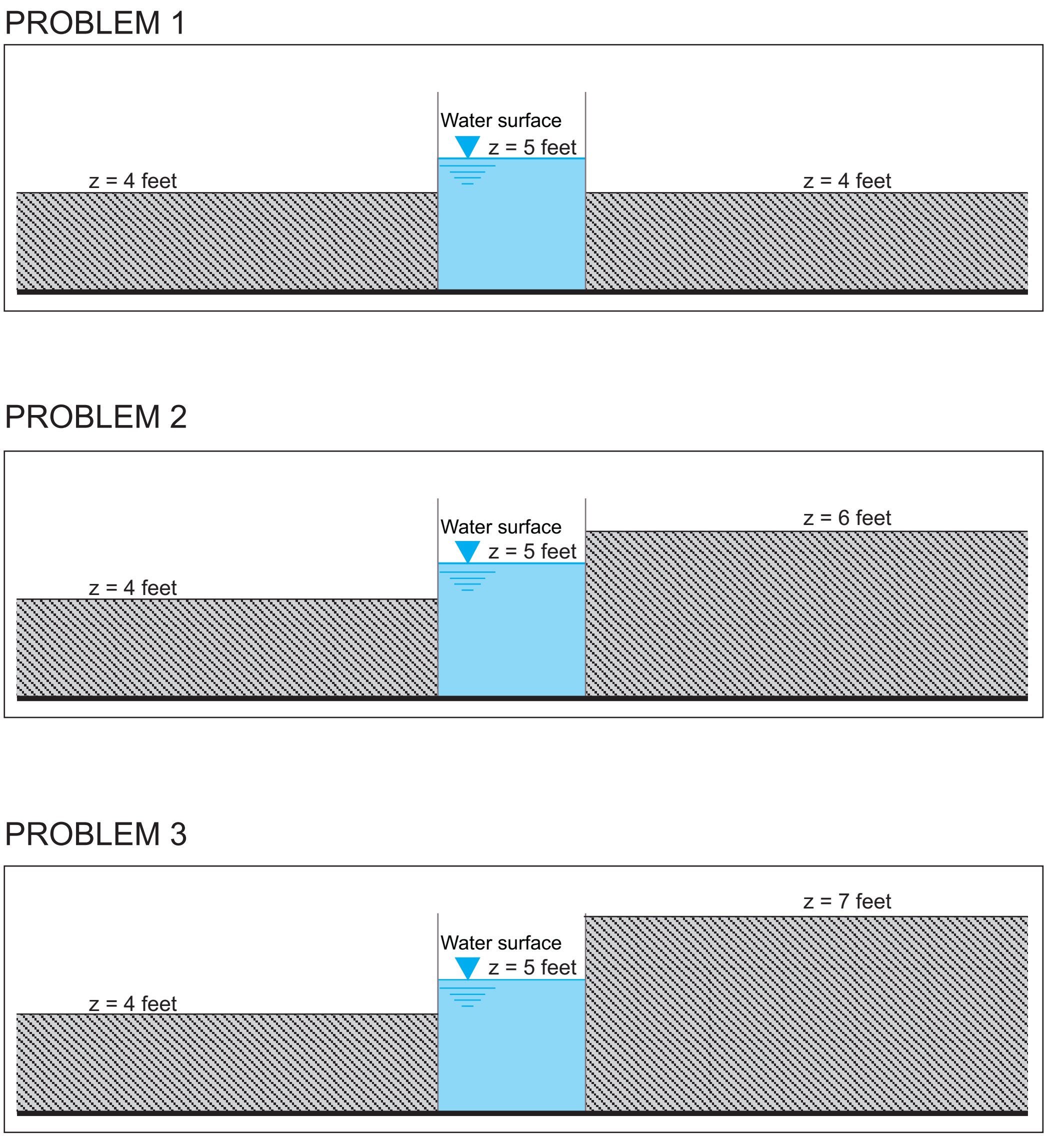


Figure 7. Cross sections of channel and aquifer for three theoretical problems (looking downstream).

Vertical-Flow and Reach-Transmissivity Results in a Field Model

The second part of the verification of the modified MODBRANCH program consisted of comparing results obtained with the modified program (using the reach-transmissivity relation) to results obtained with the original MODBRANCH program, which uses the vertical-flow relation. Verification of the original version of MODBRANCH (Swain and Wexler, 1996) included a model of a 2-mi reach of the L-31N Canal in Miami-Dade County. This same problem was analyzed using the modified MODBRANCH program and the results were compared to those obtained by the original program. The intent was to duplicate the results of Swain and Wexler (1996).

The model grid for this problem consisted of nine rows, nine columns, and one layer (fig. 8); grid spacing was chosen based on the location of ground-water monitoring wells. The aquifer top elevation was 8 ft above sea level, except directly beneath the canal where the aquifer top was set to the elevation of the canal invert; the aquifer bottom elevation was 52 ft below sea level. The hydraulic conductivity was specified as 40,000 ft/d, the confined storage coefficient as 0.0002, and the specific yield as 0.20. Ground-water heads at the model boundary varied with time to reflect field measurements. The simulation consisted of two stress periods: the first lasting 12 hours and the second lasting 48 hours. The BRANCH time step was 15 minutes, and the MODFLOW time step was 4 hours. Measured stage at the upstream and downstream ends of the channel reach were used as boundary conditions.

The 2-mi reach of canal was divided into two branches of equal length. Based on field measurements, five geometric cross sections were defined for each branch. Manning's *n* was specified as 0.025 for the entire channel. The leakage coefficient, K′/b′, used by Swain and Wexler (1996) was 0.0009 s^{-1}; this value was obtained by dividing an estimate of the reach-transmissivity coefficient (Chin 1990) by the wetted perimeter of the canal (which was estimated to be 135 ft). The modified version of MODBRANCH requires $T/(L_R - B/2)$ and $T/(L_L - B/2)$ as input parameters; the sum of these quantities replaces $\frac{K'}{b'}B$ (not the product of K'/b' and the wetted perimeter) in the finite-difference equations. In the formulation of input for the modified MODBRANCH model, the reach-transmissivity leakage parameters were specified such that the sum of $T/(L_R - B/2)$ and $T/(L_L - B/2)$ was equal to $\frac{K'}{b'}B$. Examination of the results of Swain and Wexler (1996) reveals that the average top width of the channel is about 105 ft; use of this value yields a $\frac{K'}{b'}B$ value of 0.0945 ft/s. Aquifer heads for computation of leakage in the reach-transmissivity model were obtained from the cells in column 4, where the canal is located, so that the input parameters would correspond as closely as possible to those used in the vertical-flow model. Division of the calculated value of $\frac{K'}{b'}B$ into the two reach-transmissivity leakage parameters resulted in values of 0.04725 ft/s for both $T/(L_R - B/2)$ and $T/(L_L - B/2)$.

The performance of the modified model was evaluated by comparing stage and discharge in the middle of the channel reach and aquifer head in selected cells with the results of the original model, which uses the vertical-flow relation (fig. 9). These are the same performance criteria evaluated by Swain and Wexler (1996). The modeled canal stage using the reach-transmissivity relation is virtually identical to the modeled stage using the vertical-flow relation; the values are always within 0.01 ft of each other. However, because stage was used as the boundary condition at both ends of the channel and the length of the channel is only 2 mi, proximity to the boundary probably accounts for a substantial portion of this close agreement.

Comparison of modeled discharge in the middle of the channel reach (fig. 9) provides a more meaningful measure of model performance. The discharge calculated by the vertical-flow and reach-transmissivity models are nearly identical; the difference in discharge computed with the reach-transmissivity and vertical-flow models never exceeds 0.1 ft^3/s. These results indicate that the modified MODBRANCH program yields results similar to those of the original program.

A comparison was made between aquifer head in three selected model cells at the end of each of the two time steps (table 1). The cells that were selected for comparison between models were the same as those originally selected by Swain and Wexler (1996) for comparison of modeled and measured head. Head differences between models never exceed 0.01 ft. This difference is very small and indicates that the modified MODBRANCH model performs satisfactorily.

The net modeled leakage from the canal to the aquifer for the entire simulation was $2.223 \cdot 10^7$ ft^3 and $2.219 \cdot 10^7$ ft^3 for the vertical-flow model and the reach-transmissivity models, respectively; this is a difference of only 0.18 percent. Comparison of the ground-water budget reveals that the net discrepancy between inflow and outflow for the vertical-flow model was -0.53 percent; the reach-transmissivity model had a net discrepancy of -0.52 percent. Negative values indicate that net inflow to the modeled aquifer was less than net outflow.

An unexpected benefit of incorporating the reach-transmissivity relation into MODBRANCH was the reduced number of iterations required for convergence, as well as a corresponding run time. For an attempt to duplicate the results of Swain and Wexler (1996), the MODFLOW portion of the modified model required a total of 31 iterations compared to 56 for the original vertical-flow model.

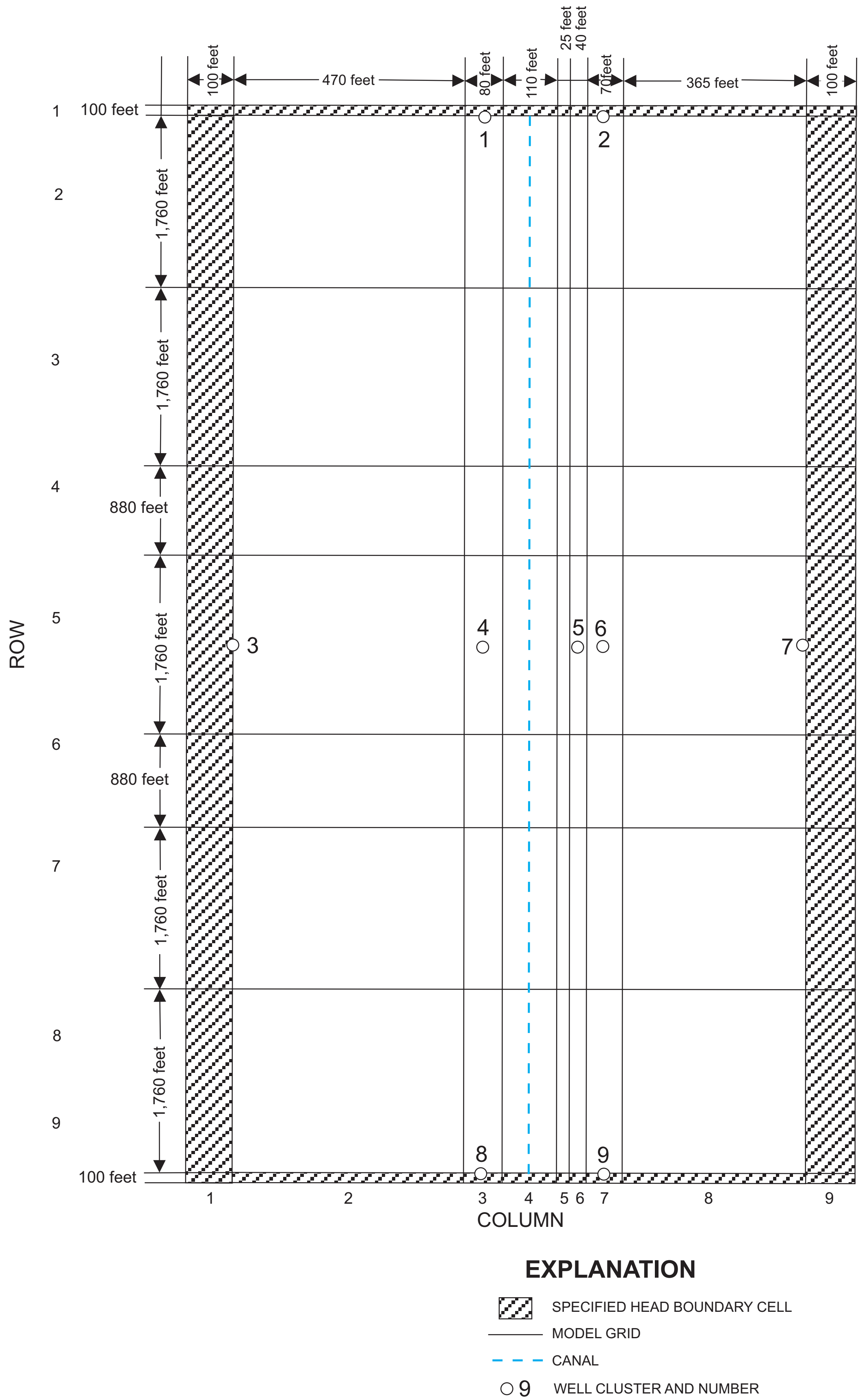


Figure 8. Model grid for Levee 31N Canal test problem (from Swain and Wexler, 1996).

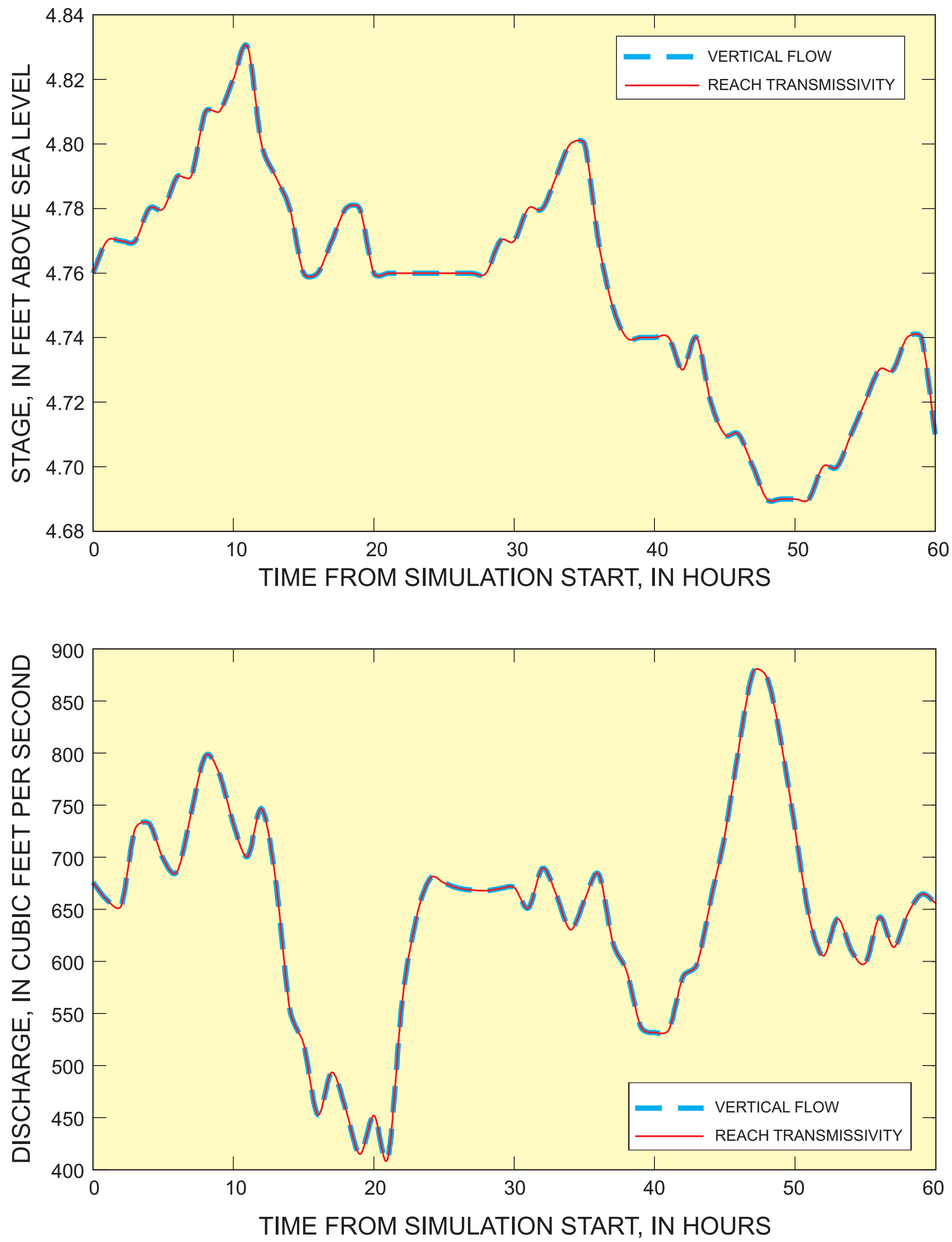


Figure 9. Comparison of stage and discharge between the vertical-flow and reach-transmissivity models.

QUANTIFICATION OF SEEPAGE BENEATH LEVEE 31N

A finite-difference numerical model was developed to quantify seepage beneath Levee 31N. The development of this model required the collection of selected field data for input and calibration. An alternate method of calculating leakage between the ground-water and surface-water systems based on reach transmissivity was incorporated into a modified version of MODBRANCH (Swain and Wexler, 1996). Model calibration and verification were performed for the vertical-flow and reach-transmissivity versions of MODBRANCH under transient conditions, and the sensitivity of the model to variations in input parameters was examined. Wetlands seepage rates obtained from the model results were compared to data obtained from seepage meter tests. Model-derived seepage rate results were used to develop an algorithm capable of estimating seepage beneath Levee 31N on the basis of head differences between selected monitoring stations.

Table 1. Ground-water head comparison for vertical flow and reach-transmissivity models

Time from simulation start (hours)	Cell location (row, column)	Vertical flow head (feet)	Reach-transmissivity head (feet)	Difference (feet)
12	5,3	4.70	4.70	0.00
	5,6	4.68	4.68	.00
	5,7	4.66	4.67	.01
60	5,3	4.58	4.58	.00
	5,6	4.55	4.55	.00
	5,7	4.53	4.54	.01

Data Collected for Model Input and Calibration

Data collected for use in the model to estimate the seepage rates beneath Levee 31N included geologic data, vertical seepage measurements, surface-water stage and discharge measurements, and continuous ground-water-level readings. The data were used to define model boundary conditions and parameters and to calibrate the model.

Geologic cores extending 62 and 42 ft below land surface were obtained during the drilling of wells G-3663 and G-3664, respectively (fig. 10 and table 2). Lithologic columns of both wells are shown in figure 11. The geologic core data were analyzed to help determine aquifer properties.

Six vertical seepage meters were installed west of Levee 31N and Canal (fig. 10). Seepage meter 1 (SM1) was installed at well G-3578; seepage meters 2 and 3 (SM2 and SM3) were installed about halfway between wells G-3577 and G-3578, with SM2 about 15 ft north of SM3; seepage meter 4 (SM4) was installed at well G-3577; and seepage meters 5 and 6 (SM5 and SM6) were installed directly east of well G-3577 and about 1,000 ft west of Levee 31N. Seepage meter SM5 is about 20 ft north of SM6. Similar meters are being used in other areas of the wetlands (Harvey, 1996) as part of the South Florida Ecosystem Place-Based Program. Seepage measurements were made during a selected number of days between August 1, 1997, and March 25, 1998. Duplicate meters were installed at two sites to evaluate the precision of measured values and the extent of local spatial variability in seepage rates.

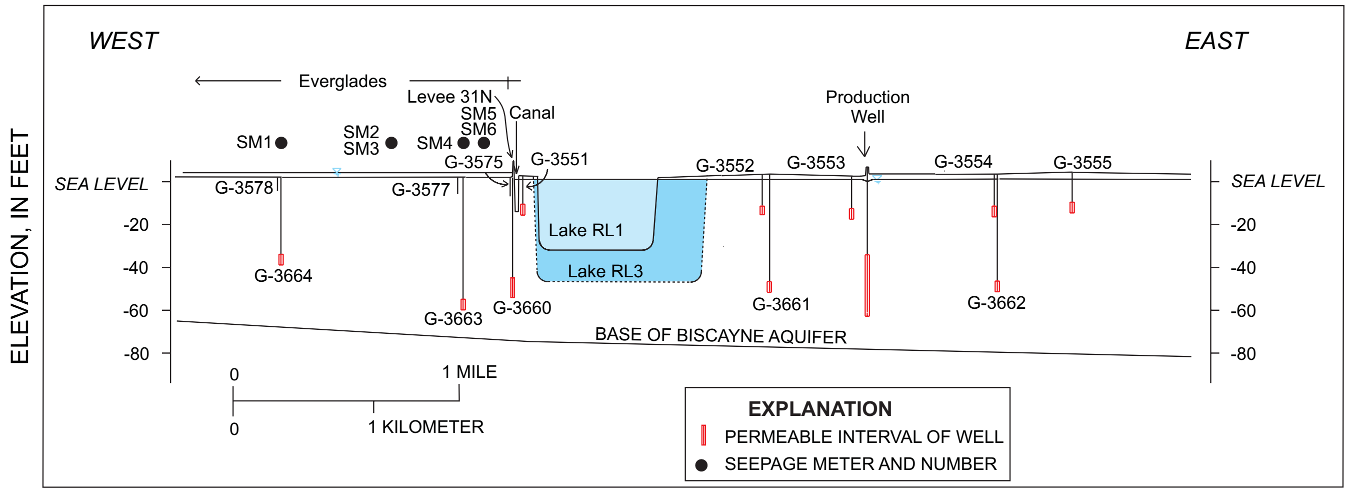


Figure 10. Transect showing location of wells and seepage meters. Well locations shown in figure 15.

Table 2. Inventory of ground-water wells drilled for the study

[Location of U.S. Geological Survey wells shown in figures 2 and 10]

Station	Latitude	Longitude	USGS Station identification number	Land surface elevation (feet above sea level)	Well depth (feet below land surface)	Casing depth (feet below land surface)	Period of record
G-618	25°45′40”	80°36′00”	254500080360001	7.40	20.0	11.0	Jan 1950 - present
G-855	25°40′38”	80°28′02”	254038080280201	7.90	20.0	10.0	Jan 1958 - present
G-3272	25°39′52”	80°32′15”	253952080321501	6.83	10.0	7.5	Jun 1983 - Feb 1985, Oct 1996 - present
G-3439	25°44′21”	80°26′02”	254421080260201	5.79	12.0	10.0	Apr 1977 - present
G-3473	25°42′48”	80°26′38”	254248080263801	8.50	20.4	20.4	Oct 1991 - present
G-3551	25°41′58”	80°29′45”	254158080294501	6.57	18.3	13.3	Apr 1994 - present
G-3552	25°41′38”	80°28′44”	254138080284401	7.41	19.4	14.4	Apr 1994 - present
G-3553	25°41′52”	80°28′21”	254152080282101	6.23	19.9	14.9	Feb 1994 - present
G-3554	25°41′52”	80°27′45”	254152080274501	7.36	20.0	15.0	Feb 1994 - present
G-3555	25°41′11”	80°27′25”	254111080272501	8.25	19.0	14.0	Mar 1994 - present
G-3556	25°42′13”	80°28′15”	254213080281501	5.14	19.1	14.1	Aug 1994 - present
G-3557	25°41′12”	80°29′42”	254112080294201	6.97	19.5	14.5	Apr 1994 - present
G-3558	25°43′34”	80°28′44”	254334080284401	7.13	19.0	14.0	Apr 1994 - present
G-3559	25°44′45”	80°29′50”	254445080295001	8.61	19.5	14.5	Apr 1994 - present
G-3561	25°40′22”	80°26′36”	254022080263601	10.44	19.0	14.0	Feb 1994 - present
G-3574	25°44′46”	80°29′55”	254446080295501	6.15	6.8	6.8	Feb 1995 - present
G-3575	25°42′06”	80°29′47”	254206080294701	5.94	9.0	9.0	Feb 1995 - present
G-3576	25°44′43”	80°30′53”	254442080305201	6.00	9.6	9.6	Mar 1995 - present
G-3577	25°42′07”	80°30′02”	254207080300201	6.00	8.0	8.0	Mar 1995 - present
G-3578	25°42′10”	80°30′48”	254210080304801	6.00	6.0	6.0	Mar 1995 - present
G-3660	25°42′09”	80°29′48”	254229080294801	--	57.0	47.0	Apr 1998 - present
G-3661	25°41′38”	80°28′44”	254138080284401	7.41	55.0	50.0	No records
G-3662	25°41′52”	80°27′45”	254152080274501	9.40	55.0	50.0	No records
G-3663	25°42′07”	80°30′02”	254207080300201	6.00	62.0	57.0	Geologic core
G-3664	25°42′10”	80°30′48”	254210080304801	6.00	41.0	36.0	Geologic core

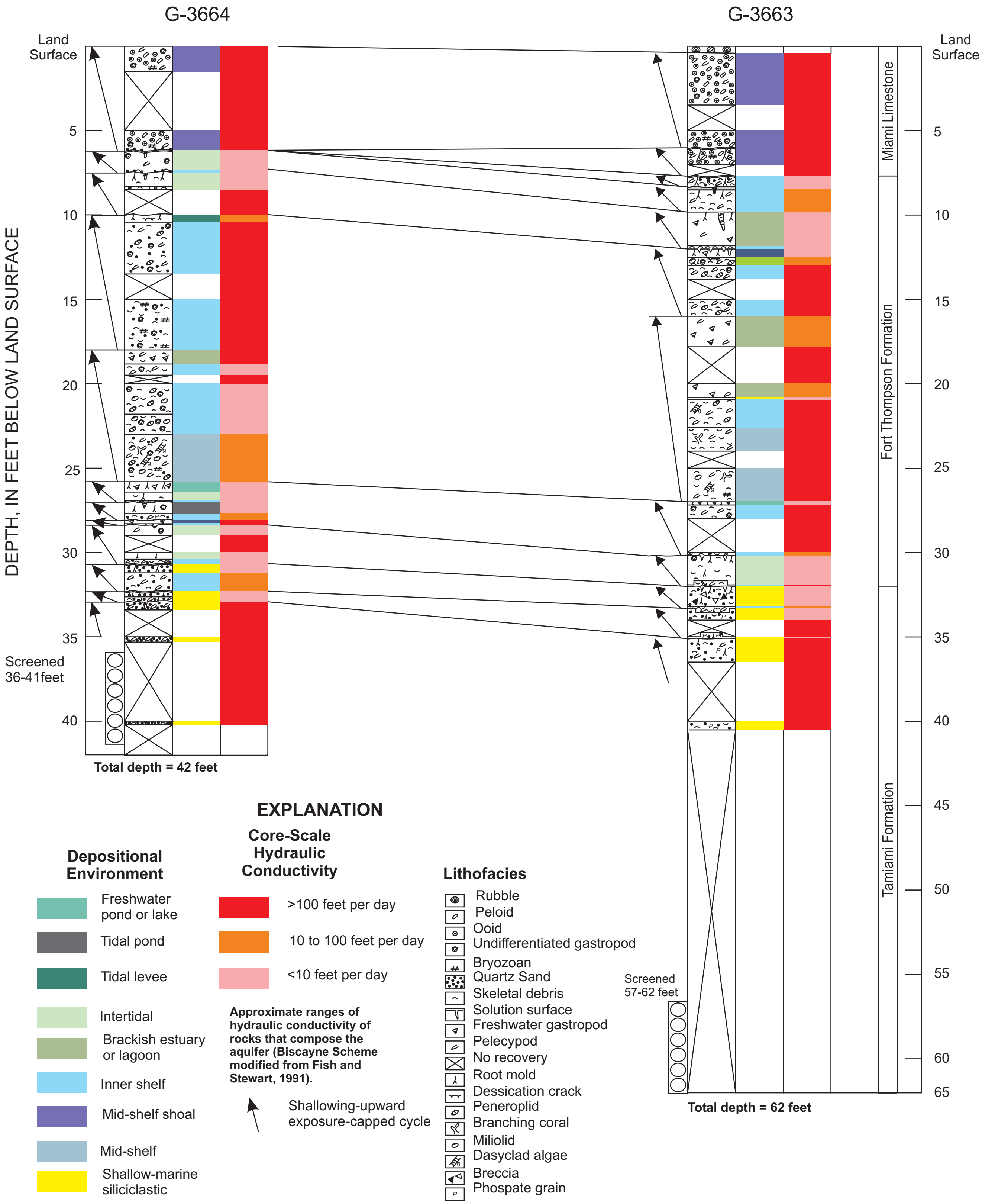


Figure 11. Lithofacies, depositional environment, and hydraulic conductivity for wells G-3663 and G-3664.

Surface-water discharge measurements were obtained at five sites in Levee 31N (L-31N Mile 3, L-31N Mile 4, and L-31N Mile 5) under various hydrologic conditions for the months of November 1996 and December 1997 (fig. 2). Flow velocities in the Levee 31N Canal are very low, generally less than 0.5 ft/s. Most of the control structures on the major canals and levees in the study area are equipped with stage recorders and discharge monitoring equipment. At some surface-water gaging stations, acoustic velocity meter (AVM) systems are used to compute discharge. These AVM gaging stations include L-31N Mile 1, L31N Mile 3, L-31N Mile 4, L-31N Mile 5, and L-31N Mile 7. The surface-water gaging stations and 13 continuous recording ground-water monitoring wells were installed along a transect that is perpendicular to, and bisected by, Levee 31N Canal (figs. 2 and 10).

Most of the ground-water wells are shallow relative to the thickness of the Biscayne aquifer. Heads recorded in these shallow wells may not be representative of the lower layers of the aquifer. Only one deep ground-water well, G-3660, is equipped with a stage recorder, which was installed in April 1998. Records of head in the lower elevations of the Biscayne aquifer are limited.

Geologic Data

Continuous core samples obtained for wells G-3663 and G-3664, located about 1.5 and 2.5 mi west of L-31N, respectively, were examined to determine hydraulic conductivity, depositional environment, and lithofacies (fig. 11). Intervals having "low" hydraulic conductivity values relative to the predominant characteristics of the extracted sample were found in each core. Support for the existence of semiconfining layers is based on hydraulic conductivity estimates and on data relating to lithology, color, fossil content, sedimentary structures, and depositional environments. The core samples from G-3663 and G-3664 were analyzed using data reported by Fish and Stewart (1991) to develop the ranges of hydraulic conductivity presented in figure 11. Based on the observed characteristics of core sample fragments, the semiconfining layers were estimated to have core-scale hydraulic conductivity values of less than 10 ft/d. However, the field-scale value is higher as a result of fractures, solution cavities, or other discontinuities in the sedimentary aquifer material that were not represented in the core sample. These local variations substantially increase the transmissivity of the aquifer, so a hydraulic conductivity of 50 ft/d was considered to be acceptable to model the semiconfining layers.

Vertical Seepage Measurements

Measured vertical seepage data indicated that water infiltrates from wetlands surface water into the ground near Levee 31N (table 3). Seepage into the ground (a decrease in the volume of water in the meter's bag) is defined as positive. Three rounds of four tests each were conducted. A number of the test results given in table 3 indicate seepage rates below the detection limit of 0.00130 ft/d. Some of the results from the first round of tests were not obtained because the plastic bags used on the seepage meters were damaged by prolonged exposure to the environment. This problem was remedied by reducing the length of each test and covering the plastic bags with an opaque fiberglass sack. Additionally, during some of the tests, the water level declined below the operational limit of the seepage meters at stations SM4, SM5, and SM6. The specific tests where this condition occurred are noted in table 3. In effect, those tests had a shorter duration because the measured changes in water volume correspond to lengths of time shorter than the entire test period. Therefore, the actual seepage rates are higher than the values calculated and presented.

To quantify the amount of water gained or lost during the process of attaching and detaching the bag, several 1-minute seepage meter tests were conducted. On average, 10 mL (milliliters) was lost during these tests; this loss of water evinced little variation from one test to another. Based on the results of the 1-minute tests, the change in water volume used in calculations for all tests was reduced from the actual measured value by 10 mL.

Seepage rates increase with proximity to Levee 31 (fig. 12, table 3). During all of these tests, the stage in the L-31N Canal and the ground-water head east of Levee 31N were lower than the water table in the wetlands west of the levee. This head differential results in the eastward flow of water beneath Levee 31N. A similar phenomenon has been observed at a site to the north, along Levee 30 (R.S Sonenshein, U.S. Geological Survey, written commun., 2000).

Table 3. Seepage measurement data

[N/A, not applicable. Location of stations shown in figure 10. Shaded seepage meter values are below detection limit of 0.00130 ft/d]

Test period	Total hours	Seepage in feet per day at selected stations					
		SM1	SM2	SM3	SM4	SM5	SM6
Test round 1							
08-01-97 to 08-17-97	384	0.00045	N/A	N/A	N/A	N/A	N/A
08-17-97 to 08-25-97	192	.00134	0.00024	0.00045	N/A	N/A	N/A
08-25-97 to 09-02-97	192	.00099	.00006	.00085	N/A	N/A	N/A
09-02-97 to 09-11-97	215	.00106	N/A	.00051	N/A	N/A	N/A
Test round 2							
10-09-97 to 10-16-97	164	.00196	.00073	.00051	0.00232	0.00247	0.00393
10-16-97 to 10-23-97	168	.00104	.00099	.00124	.00179	.00172	.00132
10-23-97 to 10-30-97[1]	168	.00157	.00044	.00086	.00146	.00261	.00212
10-30-97 to 11-04-97[1]	123	.00117	.00190	.00202	.00257	.00457	.00417
Test round 3							
02-10-98 to 02-25-98	357	.00021	.00053	.00021	.00106	.00159	.00166
02-25-98 to 03-03-98[2]	147	.00048	.00067	.00058	.00140	.00284	.00225
03-03-98 to 03-10-98	165	.00097	.00115	.00106	.00195	.00167	.00166
03-10-98 to 03-25-98	360	.00049	.00062	.00043	.00089	N/A	N/A

[1]The water level declined below the operational limit of the seepage meter at stations SM4, SM5, and SM6. The actual seepage rates are greater than the measured values reported in this table.

[2]The water level declined below the operational limit of the seepage meter at station SM4. The actual seepage rate is greater than the measured value reported in this table.

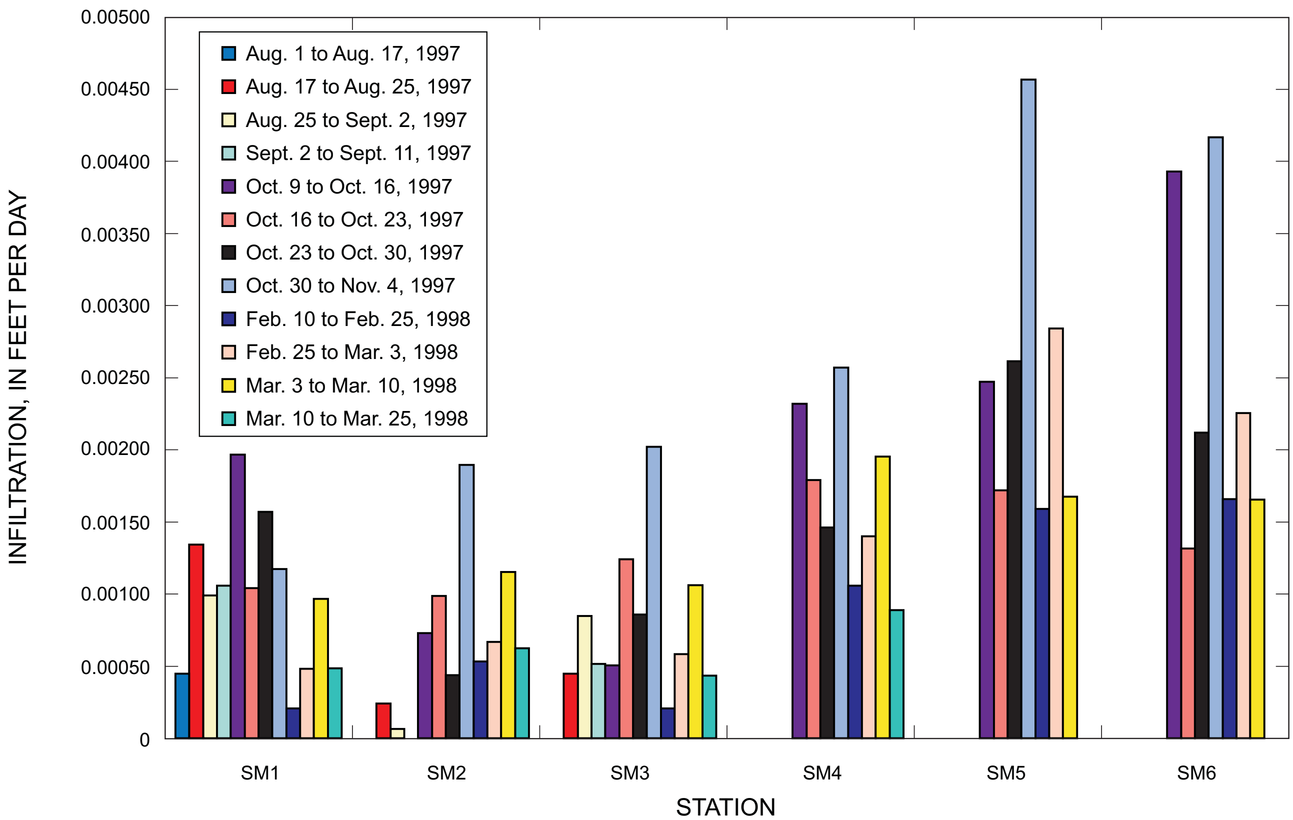


Figure 12. Seepage meter test results.

Table 4. Description of surface-water sites used in the study to determine stage and discharge

[USGS, U.S. Geological Survey; SFWMD, South Florida Water Management District; --, not applicable]

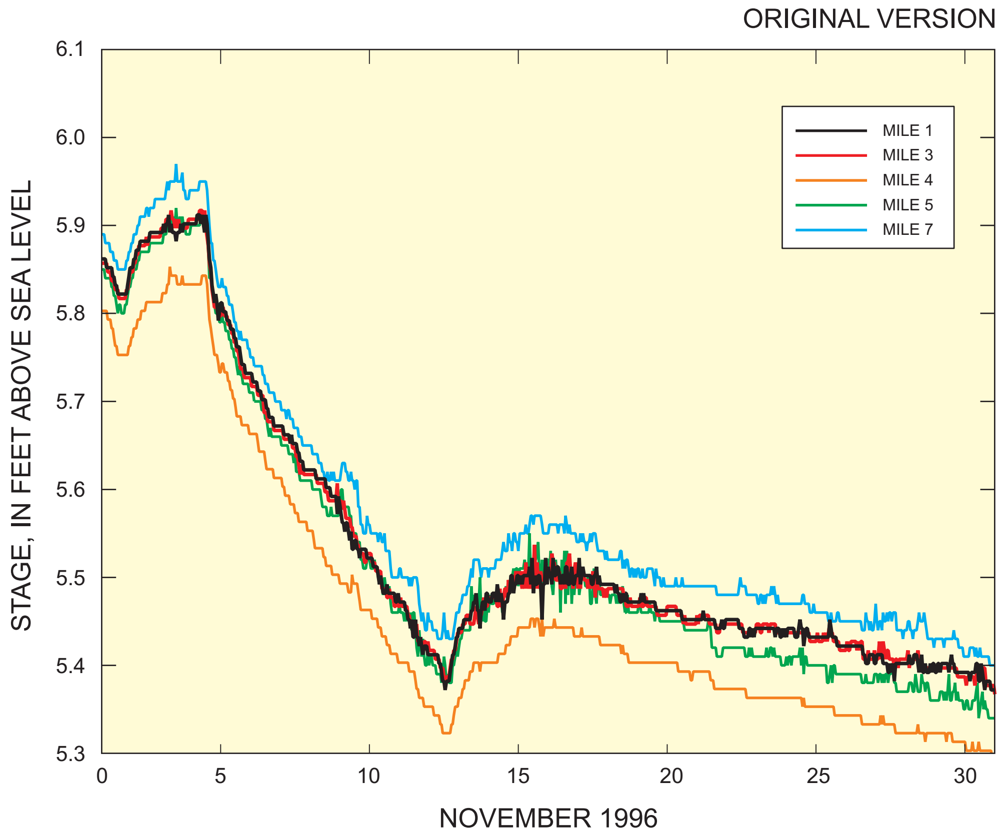
Station	Latitude	Longitude	Agency	USGS station identification number	Period of record
Surface-water stations					
L-31 Mile 1	254453	802953	USGS	02290764	November 1989 - present
L-31 Mile 3	254302	802950	USGS	02290765	February 1992 - present
L-31 Mile 4	254206	802946	USGS	02290766	June 1994 - present
L-31 Mile 5	254109	802950	USGS	02290767	June 1994 - present
L-31 Mile 7	253947	802954	USGS	02290768	June 1994 - present
Control structures					
G-69	254540	803342	SFWMD	--	March 1988 - present
G-119	254541	802838	SFWMD	--	July 1988 - present
G-211	253930	802953	SFWMD	--	October 1991 - present
S-24	254539	802955	SFWMD	--	No records
S-24A	254308	802952	SFWMD	--	No records
S-334	254541	803009	SFWMD	--	March 1988- present
S-335	254633	802859	SFWMD	--	March 1988 - present
S-336	254540	802949	SFWMD	--	March 1988 - present
S-338	253937	802850	SFWMD	--	December 1992 - present

Canal Stage and Discharge

Canal stage and discharge data were used in model calibration. Additionally, some stage data from surface-water stations at the perimeter of the modeling area were used to determine ground-water head boundaries. A description of the surface-water stations and control structures used to determine stage and discharge are given in table 4.

Examination of existing stage records from the five surface-water gaging stations in L-31N Canal has raised some questions about the accuracy with which their elevations have been surveyed. The recorded stage at all five stations for the month of November 1996 is shown in figure 13; the relative stages among the stations are typical of the entire data set. As indicated in figure 13, L-31N Mile 4 routinely records a lower stage than L-31N Miles 3 and 5, which led to the belief that surveying errors may have occurred in determining the elevations of the stations. A resurvey of all five stations, conducted by James Beadman and Associates, Inc., in December 1997 indicated that the elevations of the stations were all slightly lower than previously recorded; however, the difference between the original elevation and the resurveyed elevation was not constant for all stations (table 5). The canal stage for November 1996 using the adjusted elevations shows a substantial reduction in stage variation compared to the original measurements. The stage at some stations is still slightly lower than at other stations farther downstream, but the differences are small and do not exceed the precision of the recording instruments. Additionally, the recorded stage at L31N Mile 4 is no longer substantially lower than the stage at L31N Miles 3 and 5. The results of the resurvey were used to adjust historical stage records at the surface-water gaging stations.

Canal cross-section data are available from construction drawings or field surveys. However, substantial quantities of sediment may have been deposited in a canal since construction or the most recent field survey. A summary of cross-sectional data for the primary and secondary canals in the study area is presented in table 6.



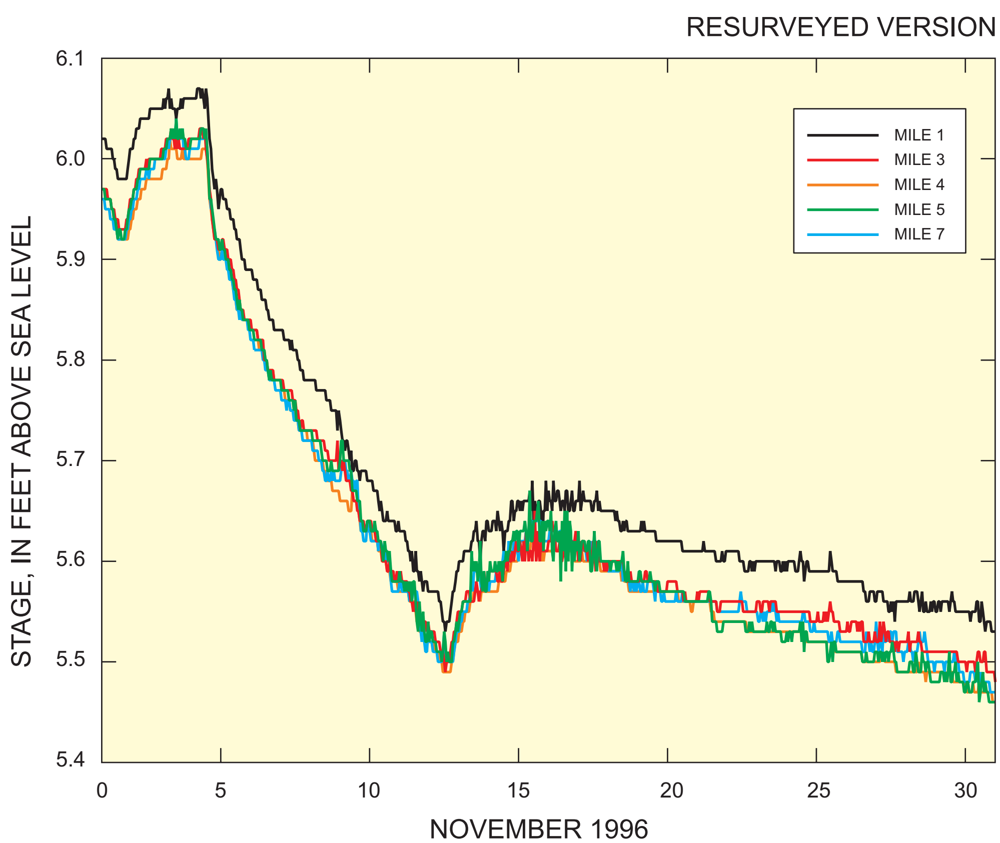


Figure 13. Recorded stage for the surface-water gaging stations in the L-31N Canal (November 1996) and adjusted for resurvey (November 1997). The data in the upper graph are the original recorded measurement. The data in the lower graph are adjusted for resurvey.

Table 5. Original and resurveyed elevations of measuring points from the surface-water gaging stations

[The resurvey was conducted in November 1996. USGS, U.S. Geological Survey; values in feet above sea level]

Station	USGS measuring point	Resurveyed measuring point	Elevation change
L-31N Mile 1	9.207	9.049	-0.158
L-31N Mile 3	9.791	9.678	-.113
L-31N Mile 4	11.748	11.581	-.167
L-31N Mile 5	11.383	11.263	-.120
L-42N Mile 7	11.782	11.712	-.070

Table 6. Canal cross-sectional data

[Construction drawings provided by the U.S. Army Corps of Engineers]

Canal	Origin and direction	Distance from origin (feet)	Bottom elevation (feet)	Bottom width (feet)	Side slopes	Data source
C-1W	L-31N junction, eastward	0 - 14,150 14,150 - 24,850	-7 -7	20 30	1:1 1:1	06-11-62 as-builts 06-11-62 as-builts
C-4 (Tamiami)	L-30/L-31N junction, eastward	0 - 16,200 16,200 - 17,200 17,200 - 21,200	-5 -2.5 -4.5	10 10 5	1:1 2:1 1:1	1976 construction drawings 3-86 survey 3-86 survey
L-29 (Tamiami)	L-30/L-31N junction, westward	0 - 1,400 1,400 - 14,600 14,600 - 15,900	-15 -7 -15	10 60 10	1:2 1:2 1:2	10-75 construction drawings 10-75 construction drawings 10-75 construction drawings
L-30	C-4/L-29 junction, northward	0 - 52,000	-10	40	2:1	P.V., Supp. 56; REDI maps
L-31N	C-4/L-29 junction, southward	0 - 56,412	-12	60	1:1	01-79 as-builts

Ground-Water Levels

Most of the ground-water wells are shallow relative to the thickness of the Biscayne aquifer. As a result, there is uncertainty as to whether the water levels recorded by shallow wells are representative of the lower layers of the aquifer. Only one deep ground-water well, G-3660, is equipped with a stage recorder, which was installed in April 1998. Water-level records in deeper parts of the aquifer are therefore limited. A comparison of the available data for well G-3660 and nearby shallow well G-3575 reveals that the water level is generally lower in well G-3660 (fig. 14).

The other deep wells (G-3661 to G-3664) are not equipped with stage recorders. However, tape-down measurements were made in wells G-3663 and G-3664 at approximately monthly intervals from March to December 1998. Wells G-3663 and G-3664 are located directly adjacent to shallow monitoring wells G-3577 and G-3578, respectively; a comparison of water-level measurements is presented in table 7. Water-level differences between wells G-3578 and G-3664 usually were nearly identical, whereas water levels in wells G-3577 and G-3663 typically differed by 0.2 to 0.3 ft. Differences in shallow and deep water levels seem to be greater in the area directly west of Levee 31N than elsewhere. Whether this effect is caused by the presence of the levee or conditions farther eastward is unknown. However, the presence of substantial differences between shallow and deep ground-water levels at some sites support the existence of semiconfining layers within the Biscayne aquifer.

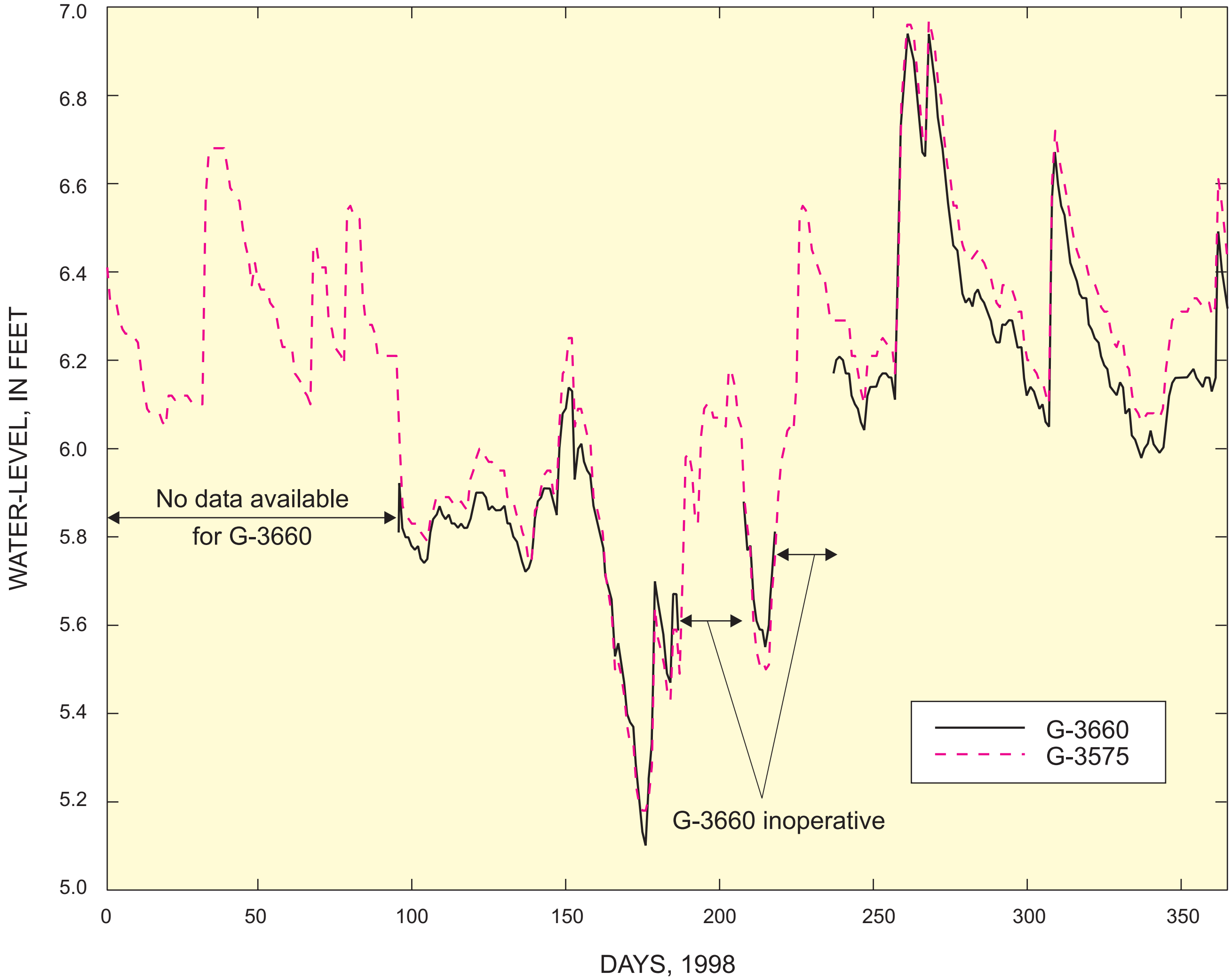


Figure 14. Water-level comparison between wells G-3575 and G-3660 for 1998.

Table 7. Comparision of water levels in shallow and deep wells

Date of measurement	Head (feet)			
	G-3577	G-3663	G-3578	G-3664
03-03-98	6.70	6.41	6.81	6.80
03-25-98	6.82	6.56	6.91	6.91
04-07-98	6.46	6.23	6.67	6.65
05-05-98	6.55	6.22	6.75	6.73
06-01-98	6.68	6.40	6.80	6.78
07-06-98	5.70	5.68	6.14	6.13
08-03-98	5.70	6.14	6.17	6.69
09-01-98	6.60	6.35	6.66	6.65
10-06-98	7.05	6.77	7.08	7.08
11-10-98	7.02	6.77	7.06	7.04
12-10-98	6.81	6.31	6.87	6.69

Levee 31N Area Flow Model

The development of a detailed model of Levee 31N and vicinity required the construction of an extensive set of input files for MODBRANCH. These files included data specifying initial and boundary conditions in the aquifer and canals, hydraulic properties, rainfall, evapotranspiration, and well pumping rates, and parameters required for the numerical calculations performed by the model.

The ground-water portion of the model, MODFLOW, required input of aquifer transmissivity, recharge, and evapotranspiration and the aquifer discretization scheme; output values were compared to observed water-table elevations in monitoring wells. The ground-water boundary conditions consisted of a temporally variable head in the cells at the perimeter of the model; these boundary conditions were developed by interpolating among monitoring well water-levels and canal stages.

The surface-water portion of the model, BRANCH, required input of channel roughness, cross-sectional geometry, and water temperature. Additionally, the required specification of lateral inflow or outflow was addressed by the leakage calculations of MODBRANCH. Initial conditions consisted of stage and discharge in all reaches of the canal; boundary conditions consisted of stage at the northern end and discharge at the southern end of the modeled portion of the L-31N Canal.

Other data required by MODBRANCH to link the ground-water and surface-water models included parameters associated with the leakage relation; different parameters were required for the reach-transmissivity and vertical-flow relations. Additionally, each canal reach defined for BRANCH was assigned to the MODFLOW cell in which it is located. The combined model was run using both leakage options, and the complete results were evaluated through comparisons to water-table elevation in monitoring wells and canal stage and discharge.

Model Discretization and Hydraulic Properties

The study area was discretized into 49 rows and 58 columns. Grid spacing was 500 ft in the vicinity of the L-31N Canal and the West Well Field and 1,000 ft elsewhere. The cells were oriented with columns parallel to the L-31N Canal to ease preparation of the input files and to ensure that the canal was located near the center of each cell, which reduces numerical errors. The exact discretization grid is shown along with the ground-water monitoring well and surface-water station locations in figure 15. The L-31N Canal was divided into four branches bounded by the five gaging stations at miles 1, 3, 4, 5, and 7. The four canal branches contained a total of 43 segments. Manning's *n* was estimated to be 0.030 based on observed characteristics of the channel (Roberson and others, 1988). Stress periods of 1 day and 1 hour were used for MODFLOW and BRANCH, respectively.

The Biscayne aquifer was discretized into six layers based on geologic characteristics, including one layer to represent surface water in wetlands and two semiconfining layers of low-permeability limestone (fig. 16). Estimates of hydraulic conductivity ranges based on geologic data were used as a starting point for model calibration. Final values of hydraulic conductivity were established through the calibration process. The thicknesses of some layers are spatially variable to more closely match the observed hydraulic properties of the aquifer. Model layer 1 represents surface water in the wetland areas and was assigned an "equivalent hydraulic conductivity" of 3,000,000 ft/d, as determined by Merritt (1995). Swain and others (1996) report satisfactory model performance with representation of wetlands in this manner. The cells in the top layer were permitted to dry and rewet. Model layer 2 consists entirely of the Miami Limestone, which was estimated to have a hydraulic conductivity ranging from 1,000 to 5,000 ft/d. The estimated hydraulic conductivity of the two low-permeability layers (model layers 3 and 5) was 50 ft/d; model layer 3 is shallower than model layer 5. Model layer 4, almost exclusively part of the Fort Thompson Formation, was estimated to have a hydraulic conductivity of at least 20,000 ft/d, which is appropriate for this formation. Model layer 6 is the deepest layer and incorporates parts of both the Fort Thompson Formation and the Tamiami Formation; the hydraulic conductivity of model layer 6 was estimated to be at least 20,000 ft/d.

The shallow semiconfining layer (model layer 3), is horizontal throughout the modeling area and extends from 5 to 7 ft below sea level (fig. 16). The deep semiconfining layer (model layer 5) is 5 ft thick; the top of this layer is sloped from an elevation of 22.5 ft below sea level in the northwestern corner of the modeling area to 42.5 ft below sea level in the southeastern corner.

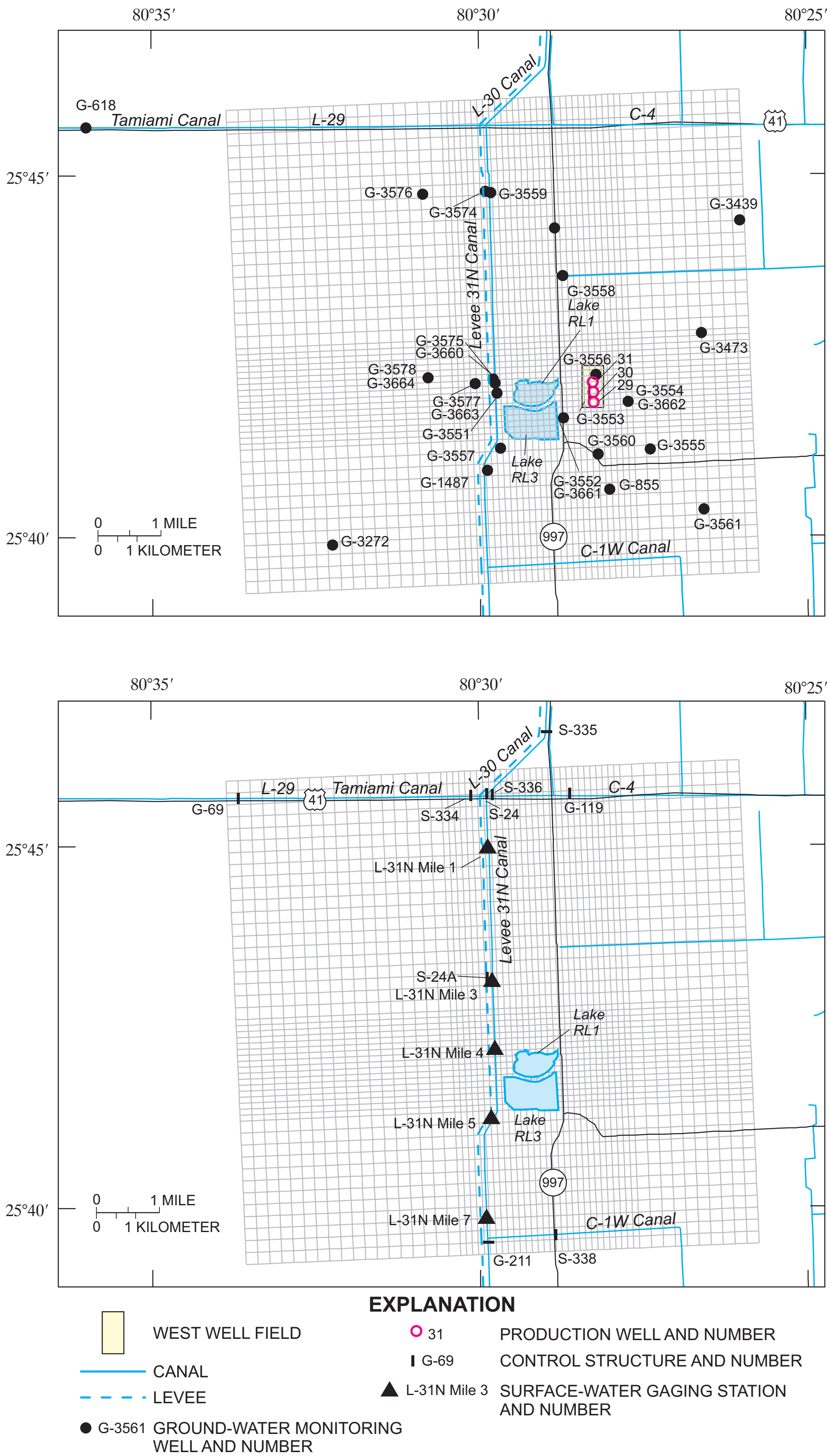


Figure 15. Model grid showing location of ground-water monitoring wells and surface-water gaging stations.

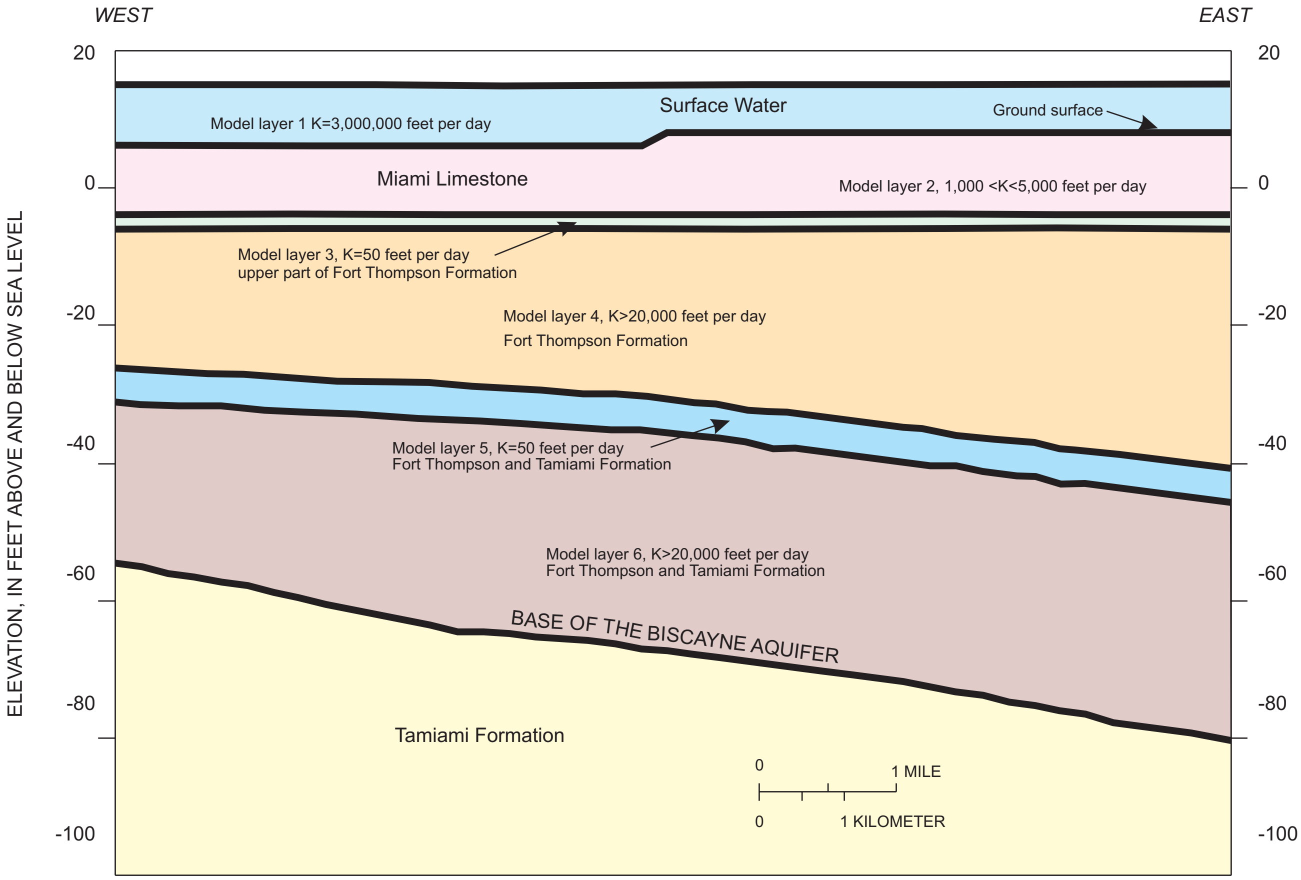


Figure 16. Model layers used to simulate hydrogeologic conditions of the Biscayne aquifer. Section extends west to east just south of Lake R13.

Initial and Boundary Conditions

Initial heads in ground-water cells were interpolated from measured values in monitoring wells (G-618, G-3439, G-3473, G-3561, and G-3572) at the beginning of the simulation period. An inverse distance method was employed in the interpolation (Sandwell, 1987). Ground-water heads at the model boundary varied with time to match field data recorded during the period covered by the simulation. Head in the boundary cells was linearly interpolated between observed values at ground-water monitoring wells and canal gaging stations along the model perimeter. The ground-water boundary cells are shown in figure 17. Ground-water boundary data were input to the model using the General-Head Boundary module of MODFLOW (McDonald and Harbaugh,

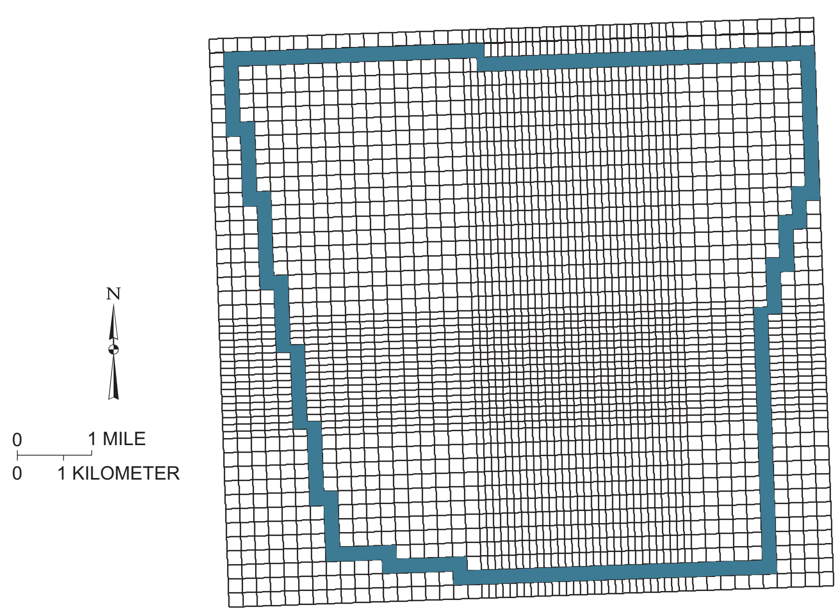


Figure 17. General head boundary cells.

1988). The surface-water boundaries consisted of specified stage at the upstream end of the channel (Mile 1) and a specified discharge at the downstream end (Mile 7). The boundary stage and discharge for each 1-hour stress period were obtained from measured values.

Recharge, Evapotranspiration, and Well Pumping

Recharge values based on precipitation were obtained from data collected at rain gages at well G-3553 and structures S-336 and S-338. The spatial distribution of rainfall within the model grid was obtained by using the discrete Thiessen method (Chin, 2000). Potential evapotranspiration data were obtained from the SFWMD station at Miami International Airport; daily averages of potential evapotranspiration from 1965 to 1995 were used as model input because data for 1996 and later years were not available. Potential evapotranspiration was assumed to be spatially constant throughout the model area. Evapotranspiration parameters were input to the model using the Evapotranspiration module of MODFLOW (McDonald and Harbaugh, 1988); on the basis of previous research, the extinction depth was specified as 20 ft on the basis of previous research (Merritt, 1995; Swain and others, 1996).

Fairbank and Hohner (1995) estimated that the average annual net recharge (rainfall minus actual evapotranspiration) to the portion of the study area east of Levee 31N ranges from 7 to 14 in. During model development, this estimate was compared to the output of the MODBRANCH model to ensure that recharge and evapotranspiration were being simulated realistically.

The three production wells of the West Well Field were represented using the Well module of MODFLOW (McDonald and Harbaugh, 1988). Water pumped by the production wells was removed from model layer 6, which corresponds to their screened interval. Daily pumping rates were specified based on operational records.

Model Results

Values of input parameters to the model were calibrated for the 1996 calendar year and confirmed by data from the 1997 calendar year. This process was performed for the original version of MODBRANCH; the version modified to use the reach-transmissivity relation was calibrated and tested for the same data set and performed adequately. The sensitivity of both models to variations in input parameters was examined, and estimates of wetland seepage provided by the model were compared to results of the seepage meter tests.

Calibration and Verification

Calibration and verification of the vertical-flow version of MODBRANCH were conducted using ground-water head, canal stage, and canal discharge data from the 1996 and 1997 calendar years. The calibration parameters used were the MODBRANCH leakage coefficient and the hydraulic conductivity of the Biscayne aquifer. The calibration process consisted of varying the calibration parameters to achieve close agreement between measured ground-water head, canal stage, and canal discharge in the study area. The calibrated values of the aquifer hydraulic conductivity are presented in table 8. The aquifer hydraulic conductivities yield an overall transmissivity of 1,400,000 ft^2/d (square feet per day) when evaluated at L-31N Mile 4; this is within the range suggested by Fish and Stewart (1991) and only slightly below an estimate of 1,800,000 ft^2/d for the region of Levee-31N located 1 to 3 mi south of the Tamiami Canal (Chin, 1990). The MODBRANCH leakage coefficient was calibrated to be 0.0009 s^{-1}, which is the same value determined by Swain and Wexler (1996) for the L-31N Canal.

Table 8. Calibrated values of hydraulic conductivity of the Biscayne aquifer

[MODBRANCH leakage coefficient = 0.0009 s^{-1}]

Model layer	Hydraulic conductivity (feet per day)
1	3,000,000
2	3,000
3	50
4	25,000
5	50
6	25,000

Calibrated results showed good correlation in ground-water heads between measured data at selected monitoring wells in the study area and the output of the vertical-flow model (fig. 18). The average errors in mean values of head and standard deviation were 0.01 and 0.13 ft, respectively, and the average coefficient of determination (R^2) was 0.79. Monitoring wells directly adjacent to the L-31N Canal had the best overall fits. In particular, the

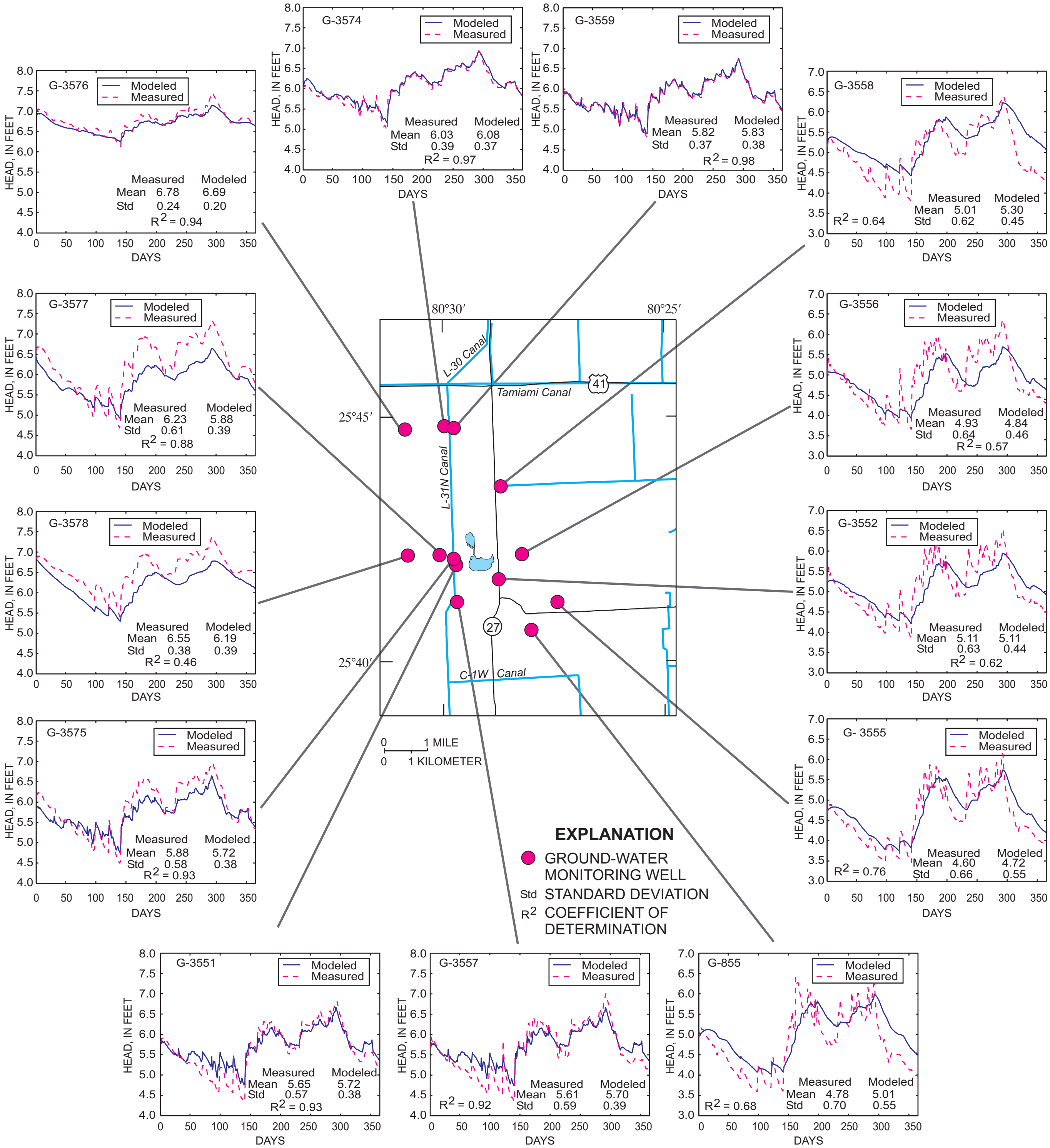


Figure 18. Measured and modeled ground-water head obtained by the vertical-flow version of MODBRANCH for the calibration run.

simulation of extreme values most closely matched measured data at these stations. The poorest fits were at the westernmost stations in the Everglades wetland areas, where annual errors in the mean ground-water head were as great as 0.36 ft. The measured standard deviation of heads almost always exceeded the modeled standard deviation; this phenomenon was most evident at stations with large variability in head.

Calibrated results for the vertical-flow model also showed good correlation between measured and modeled canal stage and discharge (fig. 19). The average error between modeled and measured stage in the canal was 0.04 ft and the mean R^2 was 0.99. However, this close agreement was probably a result of boundary effects because the simulated canal reach is only 6 mi long and has an exceptionally flat water-surface profile. The average modeled and measured standard deviation varied by less than 0.01 ft. The mean modeled and measured discharge differed by 8.6 ft^3/s, with most of the measured extreme values replicated in the model output. The mean error in the standard deviation of canal discharge was 12.7 ft^3/s, and the mean R^2 was 0.67. Some differences between modeled and measured data in the L-31N Canal can be attributed to significant errors in recorded data for all five gaging stations (U.S. Geological Survey, 1997). Average daily values (rather than hourly values) were used to ameliorate this problem, but errors in the measured data certainly exist at these stations. Under typical conditions, the accuracy of discharge measurement at the Levee 31N gaging stations is about 10 percent (M.H. Murray, U.S. Geological Survey, written commun., 1999).

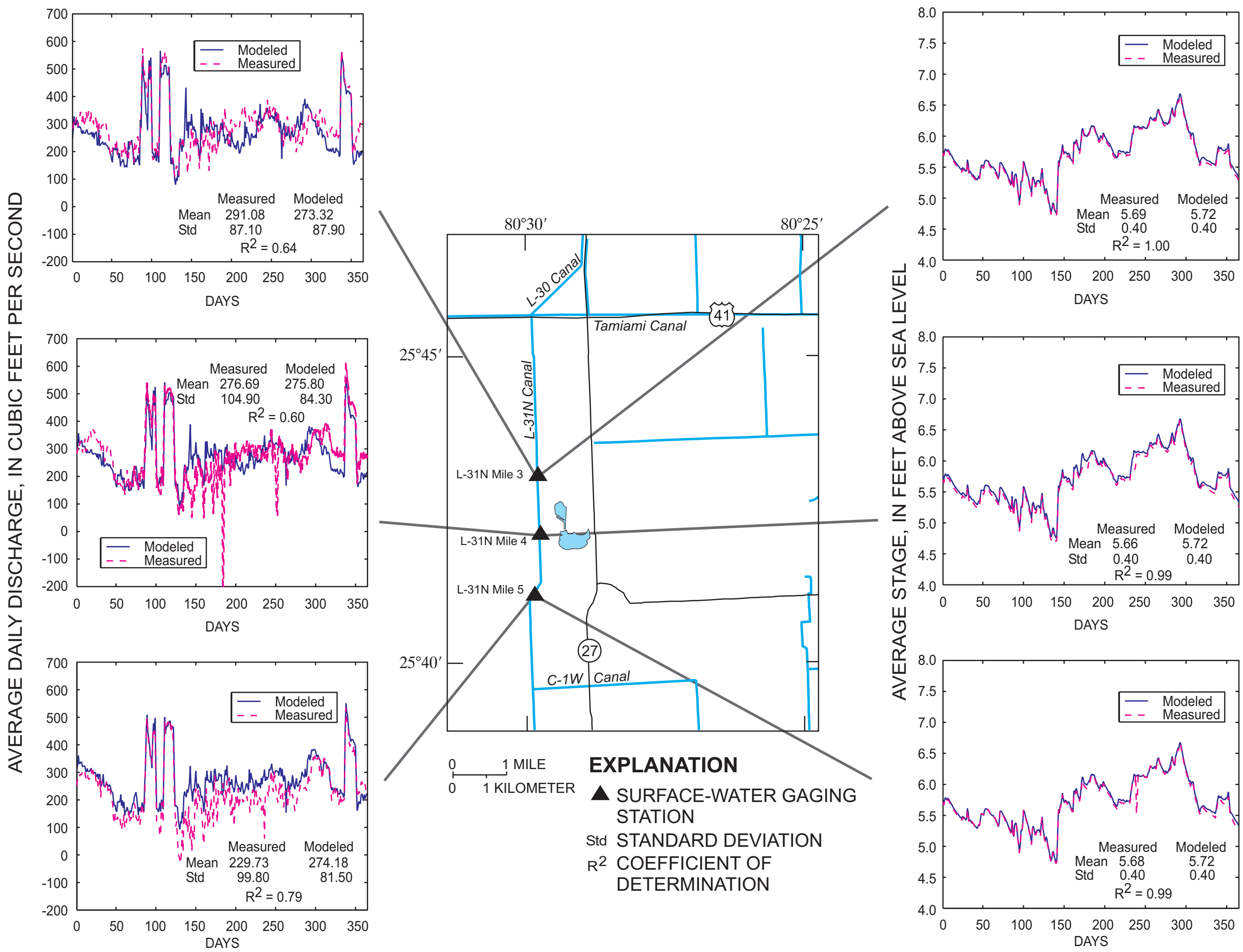


Figure 19. Measured and modeled canal stage and discharge obtained by the vertical-flow version of MODBRANCH of the calibration run.

Model verification was conducted by using the calibrated values obtained using 1996 data for a model run of the 1997 calendar year. Results of the verification run are similar to those of the calibration run (figs. 20 and 21). The 1997 data are characterized by greater short-term variability than the 1996 data, probably as a result of a greater number of extreme rainfall events. The average measured and modeled ground-water head mean and the standard deviation differed by 0.00 and 0.14 ft, respectively; the mean R^2 was 0.88. For canal stage, the average measured and modeled mean and standard deviation both differed by 0.01 ft and 0.02 ft, respectively; the average R^2 was

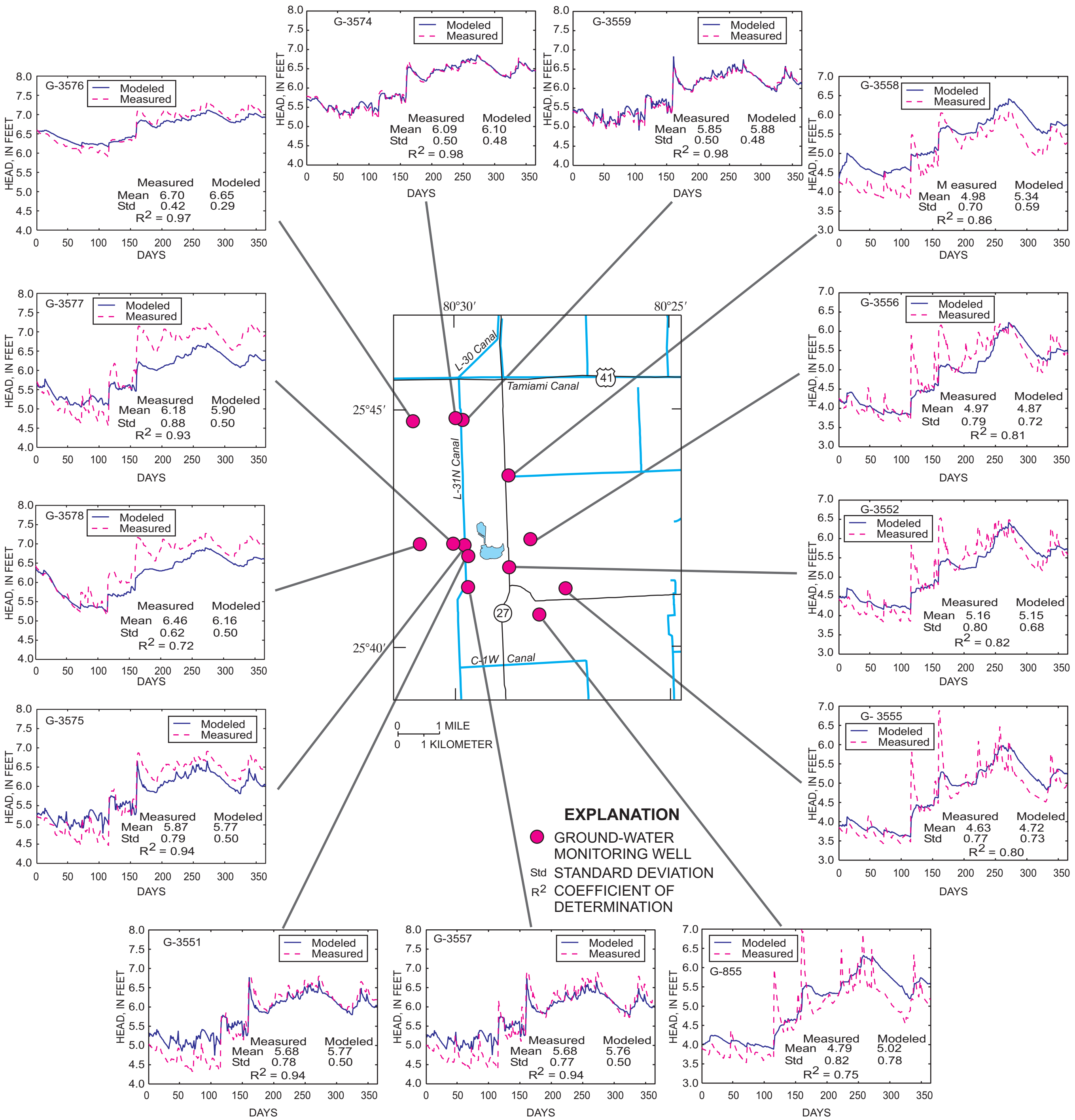


Figure 20. Measured and modeled ground-water head obtained by the vertical-flow version of MODBRANCH for the verification run.

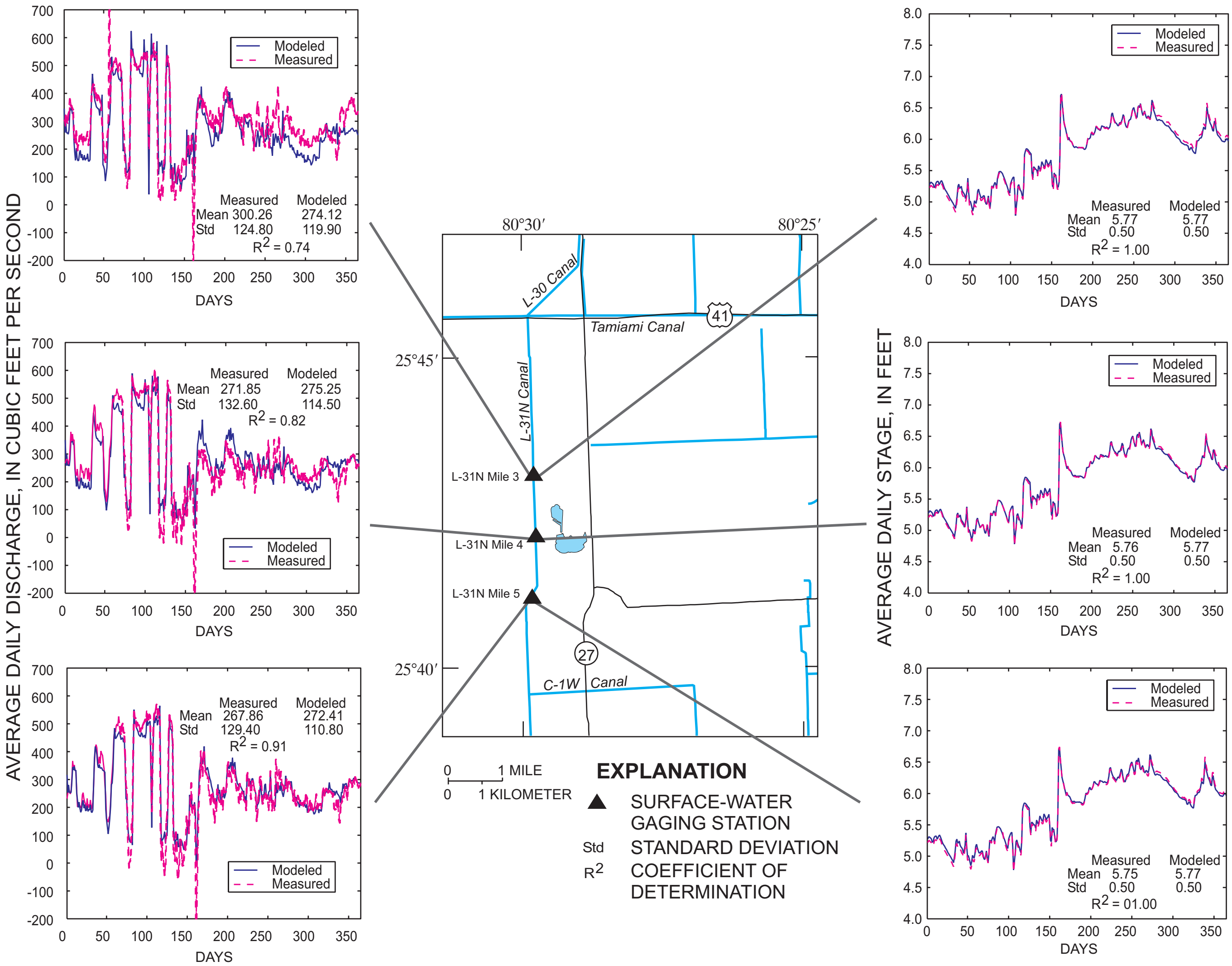


Figure 21. Measured and modeled canal stage and discharge obtained by the vertical-flow version of MODBRANCH for the verification run.

greater than 0.99. For canal discharge, the average differences between measured and modeled mean and standard deviation were 6.1 and 13.8 ft^3/s, respectively; the mean R^2 was 0.82. As with the calibration run, the modeled standard deviation of values was less than the measured standard deviation, and again, the mean values matched well. The overall fit of the calibration run was deemed sufficient to accept the calibrated values of aquifer hydraulic conductivity and the MODBRANCH leakage coefficient.

Evaluation of the Modified Version of MODBRANCH

The modified version of MODBRANCH, incorporating the reach-transmissivity leakage relation, was calibrated and verified for the same set of data as the vertical-flow version of MODBRANCH. The previously calibrated hydraulic conductivities of the Biscayne aquifer (table 8) were used in evaluating the reach-transmissivity relation to facilitate comparisons to the results obtained with the vertical-flow relation. The parameters used for calibration of the reach-transmissivity model were the leakage coefficients of the modified version of MODBRANCH. As previously noted, the modified model requires one leakage coefficient for each side of the channel. The coefficients are derived from the aquifer transmissivity and the distance from the channel to the point at which the ground-water reference head is obtained; the right and left coefficients are $T/(L_R - B/2)$ and $T/(L_L - B/2)$, respectively.

Once the aquifer transmissivity is established, all of the components of the leakage coefficients are essentially fixed except for L_R and L_L, assuming constant transmissivity. For the Levee 31N model, the transmissivity has been previously calibrated as 1,400,000 ft^2/d. The transmissivity was assumed to be constant along the entire channel because of the difficulties of calculating separate values for each channel segment, and also because of its small expected spatial variation. Therefore, calibration of the leakage coefficients consisted of selecting the most appropriate values for L_R and L_L; these distances also determined the model cell from which the ground-water reference heads were obtained. The values of L_R and L_L were selected so that the point from which the aquifer reference head was obtained was always at the center of a cell.

Several values of L_R and L_L were examined, and the results of the calibration indicated that the best data fit occurred when the ground-water reference head was obtained from the cells directly adjacent to the channel. The reach-transmissivity leakage coefficients, $T/(L_R - B/2)$ and $T/(L_L - B/2)$ both had calibrated values of 0.0363 ft/s. Calibration and verification of the reach- transmissivity model were conducted using ground-water head, canal stage, and canal discharge data from 1996 and 1997, respectively (figs. 22-25). Results were similar to those of the vertical-flow relation in MODBRANCH. The reach-transmissivity model results were usually slightly better than the vertical-flow model results in most of the study area. The exception was in the area adjacent to the L-31N Canal where results were usually slightly worse. Model performance was judged by mean values and coefficients of determination between modeled and measured values. Standard deviations were similar for both methods and large differences in results were not evident at any monitoring station. The reach-transmissivity relation was deemed to have performed adequately in the MODBRANCH program.

A reduction in the number of iterations required by the modified version of MODBRANCH was observed. The total number of iterations and the run time were reduced by about 40 percent, relative to the original version of MODBRANCH. This is approximately the same proportional decrease observed in duplication of the results of Swain and Wexler (1996).

Sensitivity Analysis

Model calibration by an objective optimization procedure was beyond the scope of this study, so a formal sensitivity analysis was not undertaken. Nevertheless, during the calibration process it became evident that the model was more sensitive to variations in aquifer conductivity than the MODBRANCH leakage coefficient. Additionally, poor selection of input parameters led to failure of the model to converge to a solution. This problem was most evident when input values of aquifer hydraulic conductivity were too small or the leakage coefficient was too large. The inability to obtain a solution for certain configurations of input parameters limited the availability of output sets for comparison as part of a sensitivity analysis.

The effect of varying input parameters was generally small during calibration of the original version of MODBRANCH. For example, reducing the leakage coefficient from 0.0009 to 0.0004 s^{-1} and the overall transmissivity of the aquifer from 1,400,000 to 560,000 ft^2/d resulted in the changes in ground-water head presented in table 9. The most substantial differences between results occurred in the vicinity of the West Well Field (near well G-3553) where the lower aquifer transmissivity led to greater drawdown from well pumping.

As previously noted, the hydraulic conductivities obtained for the vertical-flow relation were used without alteration in the reach-transmissivity model. Therefore, calibration of the reach-transmissivity model consisted only of adjusting the leakage coefficients by changing the model cell from which the reference ground-water head was obtained. Generally, assignment of the ground-water reference head to cells more distant from the channel resulted in a small increase in modeled aquifer head and canal stage. This improved the fit at stations directly adjacent to the canal, but worsened the fit everywhere else. The changes in ground-water head resulting from assignment of the ground-water reference head to the model cells, two rows away from the channel (instead of one row), are summarized in table 9 for selected wells. The simulated heads are similar; changing the distance from the centerline of the canal at which the reference ground-water head is obtained does not substantially affect the results. This would be expected because the reach-transmissivity leakage equation is theoretically applicable at any distance from the channel, provided the Dupuit-Forcheimer assumption is valid.

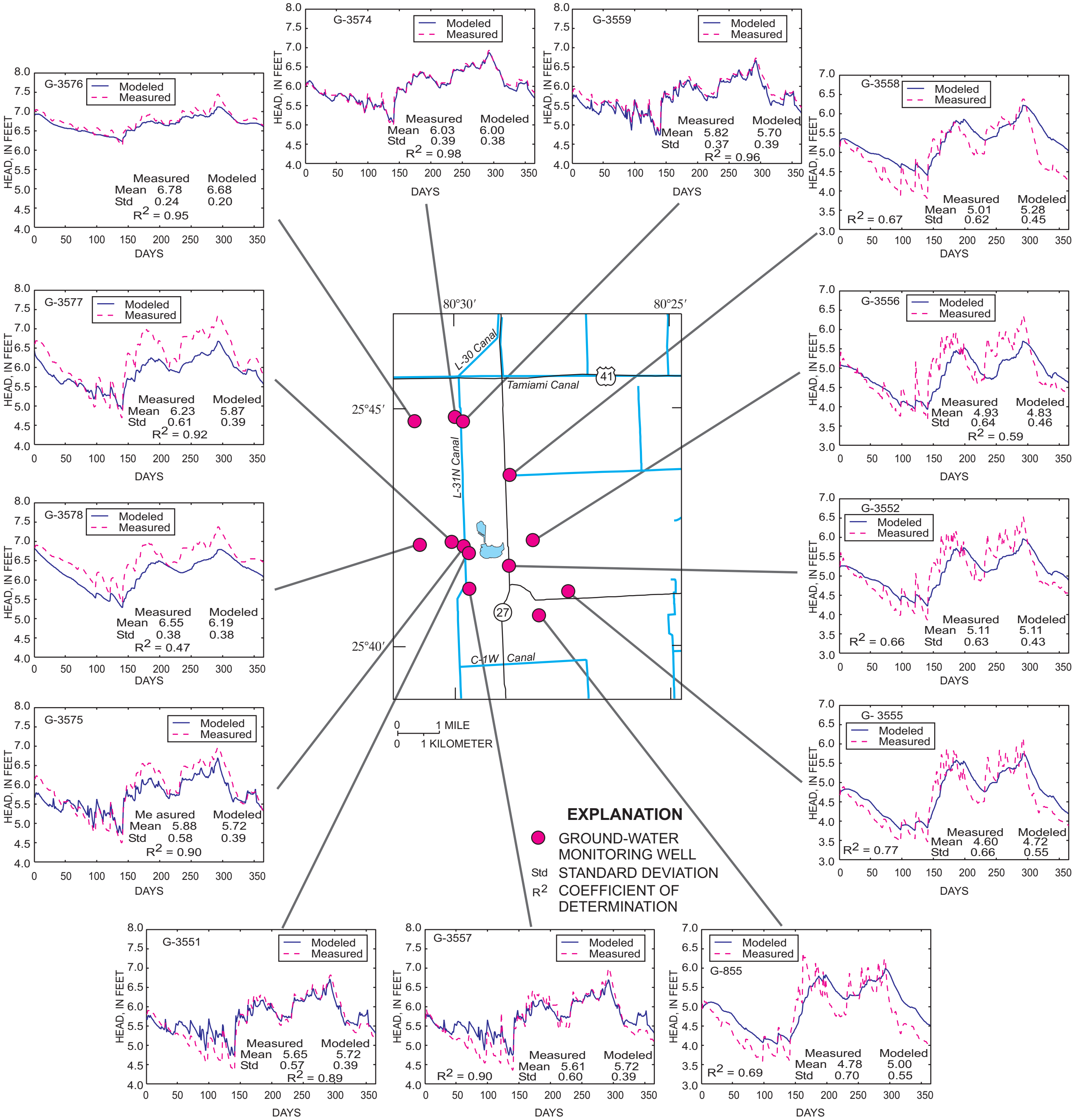


Figure 22. Measured and modeled ground-water head obtained by the reach-transmissivity version of MODBRANCH for the calibration run.

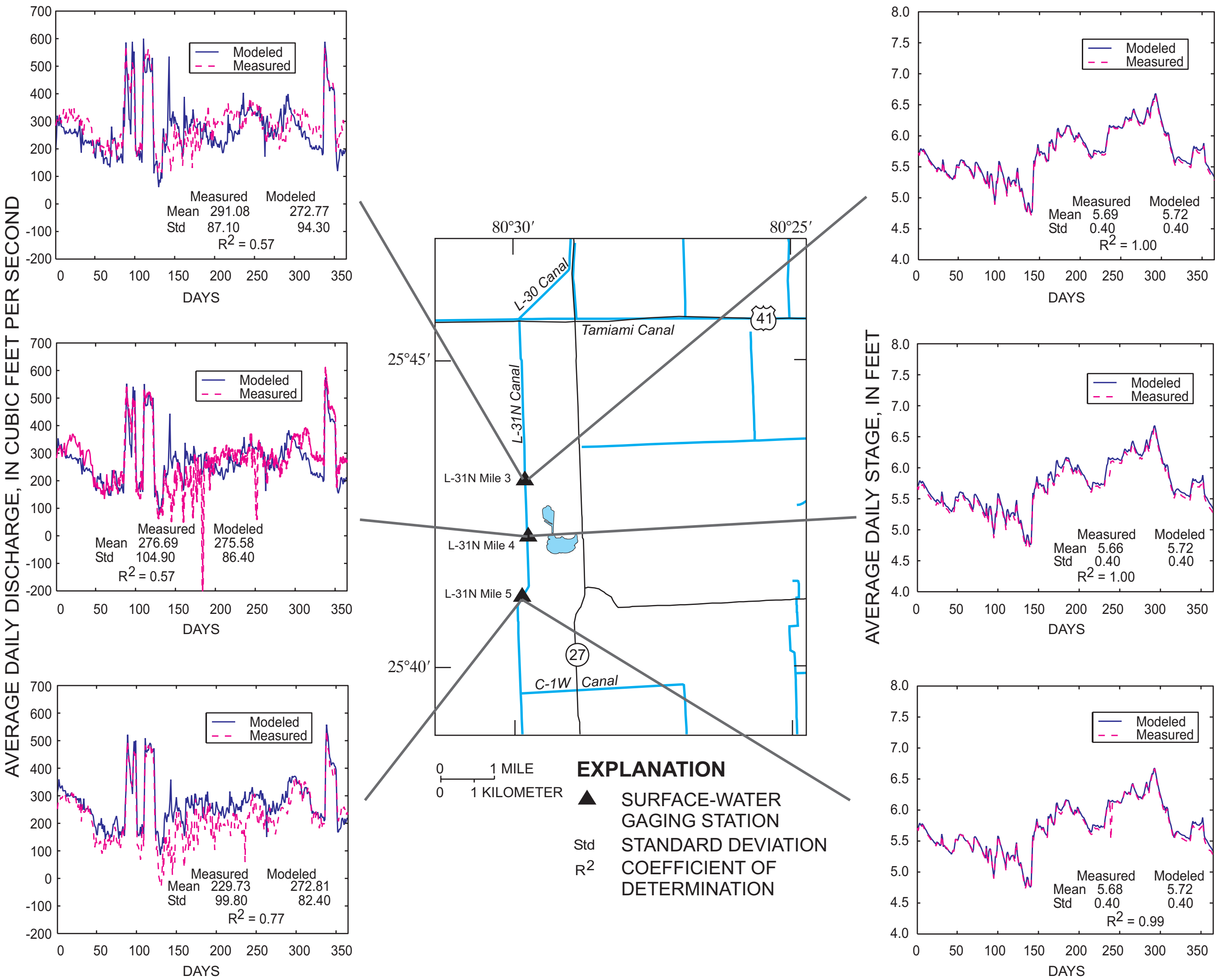


Figure 23. Measured and modeled canal stage and discharge obtained by the reach-transmissivity version of MODBRANCH for the calibration run.

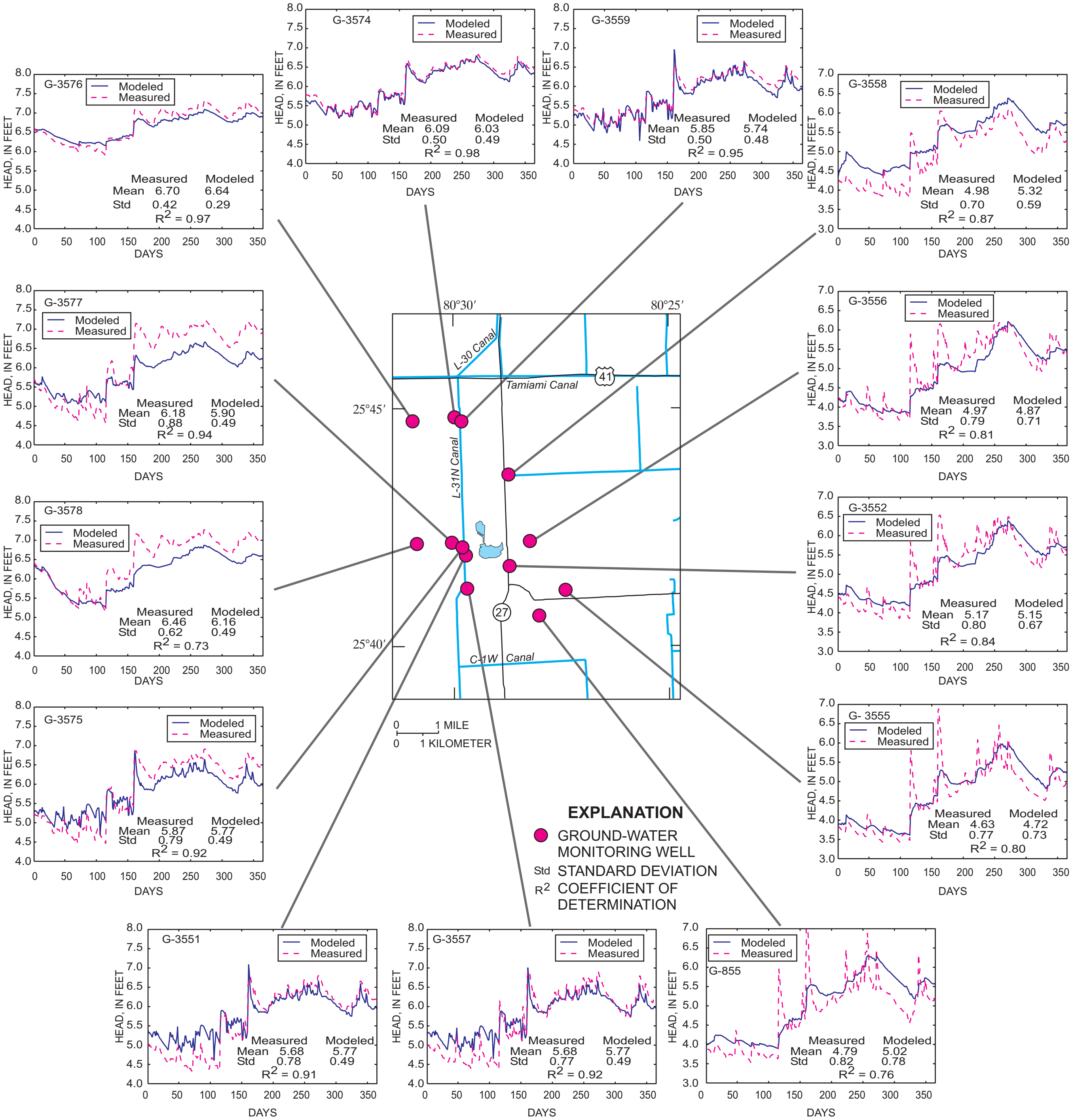


Figure 24. Measured and modeled ground-water head obtained by the reach-transmissivity version of MODBRANCH for the verification run.

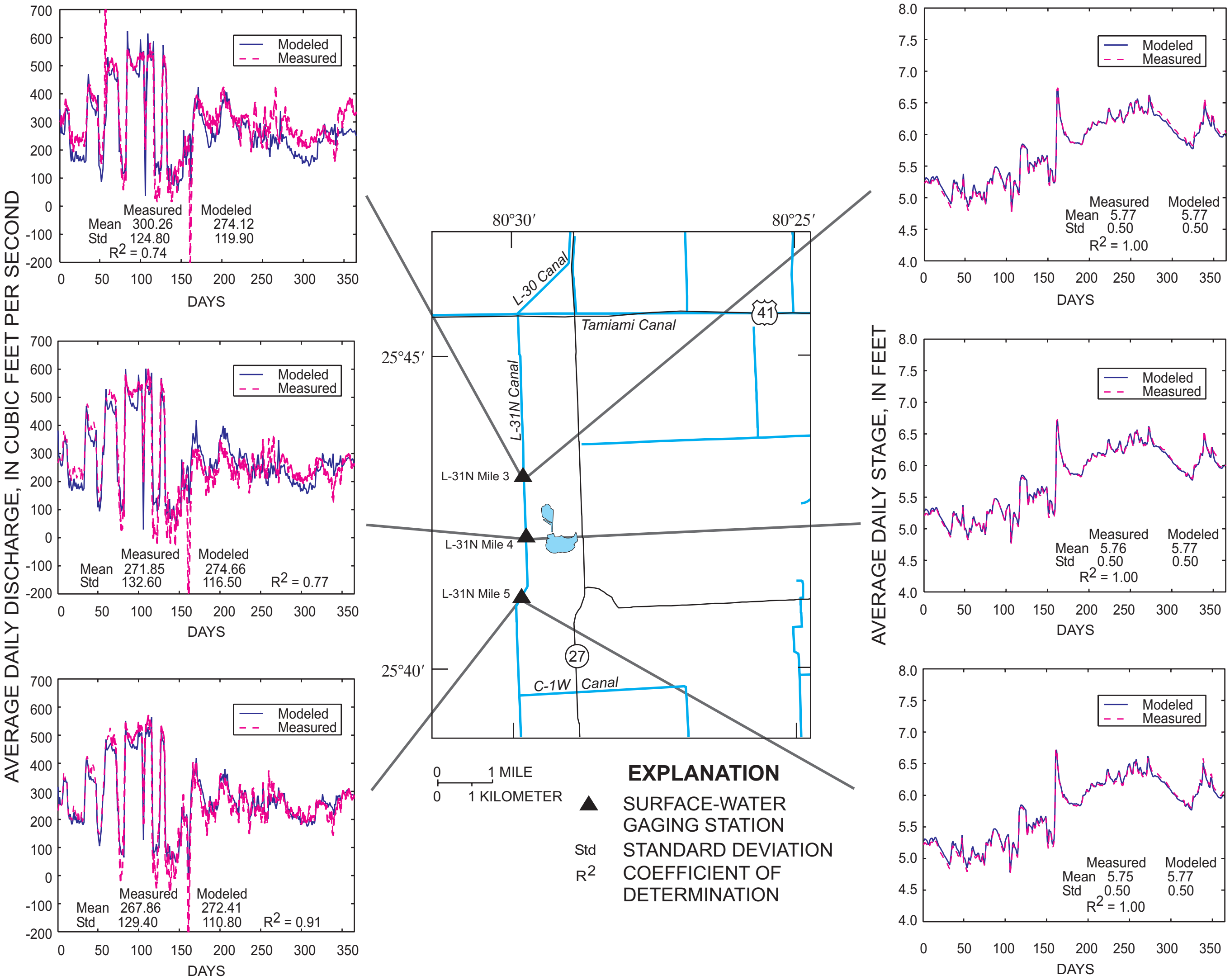


Figure 25. Measured and modeled canal stage and discharge obtained by the reach-transmissivity version of MODBRANCH for the verification run.

Table 9. Model sensitivity to changes in aquifer hydraulic conductivity and MODBRANCH leakage coefficients

[Data are for 1996. Abbreviations: T, transmissivity; ft/d, feet per day; c, leakage coefficient; s^{-1}, per second]

Station	Head (feet) based on vertical-flow model			Head (feet) based on reach-transmissivity model		
	T = 1,400,000 ft/d c = 0.0009 s^{-1}	T = 560,000 ft/d c = 0.0004 s^{-1}	Difference (feet)	Aquifer head 1 cell from channel	Aquifer head 2 cell from channel	Difference (feet)
G-3551	5.72	5.72	0.00	5.72	5.72	0.00
G-3553	4.73	4.04	.69	4.72	4.73	-.01
G-3555	4.72	4.58	.14	4.72	4.72	.00
G-3558	5.30	5.20	.10	5.28	5.31	-.03
G-3575	5.72	5.73	.00	5.72	5.73	-.01
G-3577	5.88	5.96	-.08	5.87	5.89	-.01
G-3578	6.19	6.29	-.10	6.19	6.19	.00

Numerical Model and Seepage Meter Results

The MODFLOW model calculates the flow of ground-water from every active cell to all adjacent active cells for each time step. These cell-by-cell flow data are useful when the magnitude of ground-water flow at a particular point is of interest. The pertinent cell-by-cell flow results from the calibrated reach-transmissivity version of MODBRANCH were compared to the results of the seepage meter tests. Modeled rates of flow between layer 1 (the wetlands layer) and layer 2 (the top ground-water layer) were examined for the cells that correspond to locations of seepage meters; flow rates were averaged over the period of seepage meter tests. Model results are unavailable for some test periods because some cells in the wetlands layer dried out and were converted to inactive status. Comparison of the modeled seepage rates (table 10) with the measured seepage meter results (table 3) reveals important dissimilarities. The modeled seepage rates are frequently more than 10 times greater than the measured seepage rates. However, the proportional increase in flow rates between stations as Levee 31N is approached is similar in both modeled and measured results.

The large discrepancies between measured and modeled seepage rates may be a result of underestimation in measured flow rates. Seepage meters quantify vertical flow of water within an area of only a few square feet. In contrast, the smallest cells of the computer model are 250,000 ft^2 (square feet) in area. As previously noted, the Biscayne aquifer is characterized by the presence of sinkholes, fractures, and solution cavities. Additionally, in some areas of the wetlands, up to a few feet of muck soil covers the underlying limestone, whereas in other areas the ground surface consists of bare rock. Substantial variations in the ground surface were observed during installation of the seepage meters. Vegetation was highly variable, and the thickness of the soil layer atop the underlying limestone was commonly up to three times greater at one location than at another only a few feet away. Furthermore, to function properly, the seepage meters must be installed where a substantial layer of peat or soil is present; the local hydraulic conductivity at such locations is less than in areas where the limestone aquifer is exposed at the surface.

Hydraulic conductivity is scale-dependent. The bulk hydraulic conductivity and flow rate within an area as large as one of the cells of this model may be an average of the properties of multiple smaller areas with greatly varying properties. Given the nonuniform nature of the Biscayne aquifer in the region where the seepage meters were operated, large local variability probably exists in hydraulic conductivity. The requirements for seepage-meter installation are such that they cannot be operated at sites likely to have the highest hydraulic conductivity (for example, where limestone is exposed at land surface). Consequently, the seepage meters may accurately quantify flow rates and hydraulic conductivity at their locations without providing results that are necessarily representative of large-scale characteristics of the aquifer. If seepage estimates for the wetlands area west of Levee 31N are based solely on the results of the seepage meter tests, vertical seepage may tend to be underestimated because the seepage meters can only be operated properly at sites where the lowest hydraulic conductivity is expected.

Table 10. Average vertical seepage rates calculated by the reach-transmissivity model during periods corresponding to seepage meter tests

[--, not measured]

Test period	Average seepage (feet per day)			
	Station SM1	Stations SM2, SM3	Station SM4	Stations SM5, SM6
08-01-97 to 08-17-97	0.021	0.019	0.017	0.021
08-17-97 to 08-25-97	.014	.014	.016	.024
08-25-97 to 09-02-97	.023	.024	.026	.035
09-02-97 to 09-11-97	.028	.029	.032	.044
10-09-97 to 10-16-97	.009	.013	.021	.043
10-16-97 to 10-23-97	.011	.016	.024	.044
10-23-97 to 10-30-97	.010	.016	.027	.045
10-30-97 to 11-04-97	.011	.018	.033	.048
02-10-98 to 02-25-98	.015	.026	--	--
02-25-98 to 03-03-98	.020	.046	--	--
03-03-98 to 03-10-98	.022	.049	--	--
03-10-98 to 03-25-98	.027	.043	--	--

Algorithm for Estimating Real-Time Seepage

Estimations of seepage losses beneath Levee 31N can be used to assess the effect of water-management practices, such as gate operation. However, developing a complete set of MODBRANCH input data to represent conditions that change daily is impractical. A simple algorithm to estimate seepage, based on a minimum of input parameters, is more useful for hydrologic management activities.

An effort was made during the development of the seepage algorithm to ensure that the input parameters were available on a real-time basis. Ground-water heads and canal stage are continually recorded and, in some cases, are available instantaneously. Head gradients were expected to be the dominant influence on ground-water flow. Other parameters, such as evapotranspiration and rainfall, are more difficult to measure and predict and were believed to have substantially less effect on ground-water seepage rates. Additionally, the effects of evapotranspiration and rainfall are implicitly reflected in ground-water and surface-water levels. Therefore, it was determined that the algorithm should predict seepage on the basis of head differential between monitoring stations.

The MODBRANCH model was run for the 1996 and 1997 calendar years with the input values previously calibrated. Data from 1996 were used for calibration of the seepage algorithm, and data from 1997 were used for verification. The reach-transmissivity version of MODBRANCH was employed because it provided slightly better simulation of aquifer heads than the vertical-flow version. The eastward cell-by-cell flows across the column boundaries directly west of Levee 31N were summed by layer at daily intervals between the gaging stations at Miles 1 and 7 (fig. 26). Model layers 2, 4, and 6 conveyed the majority of flow; the cells in layer 1 along Levee 31N were

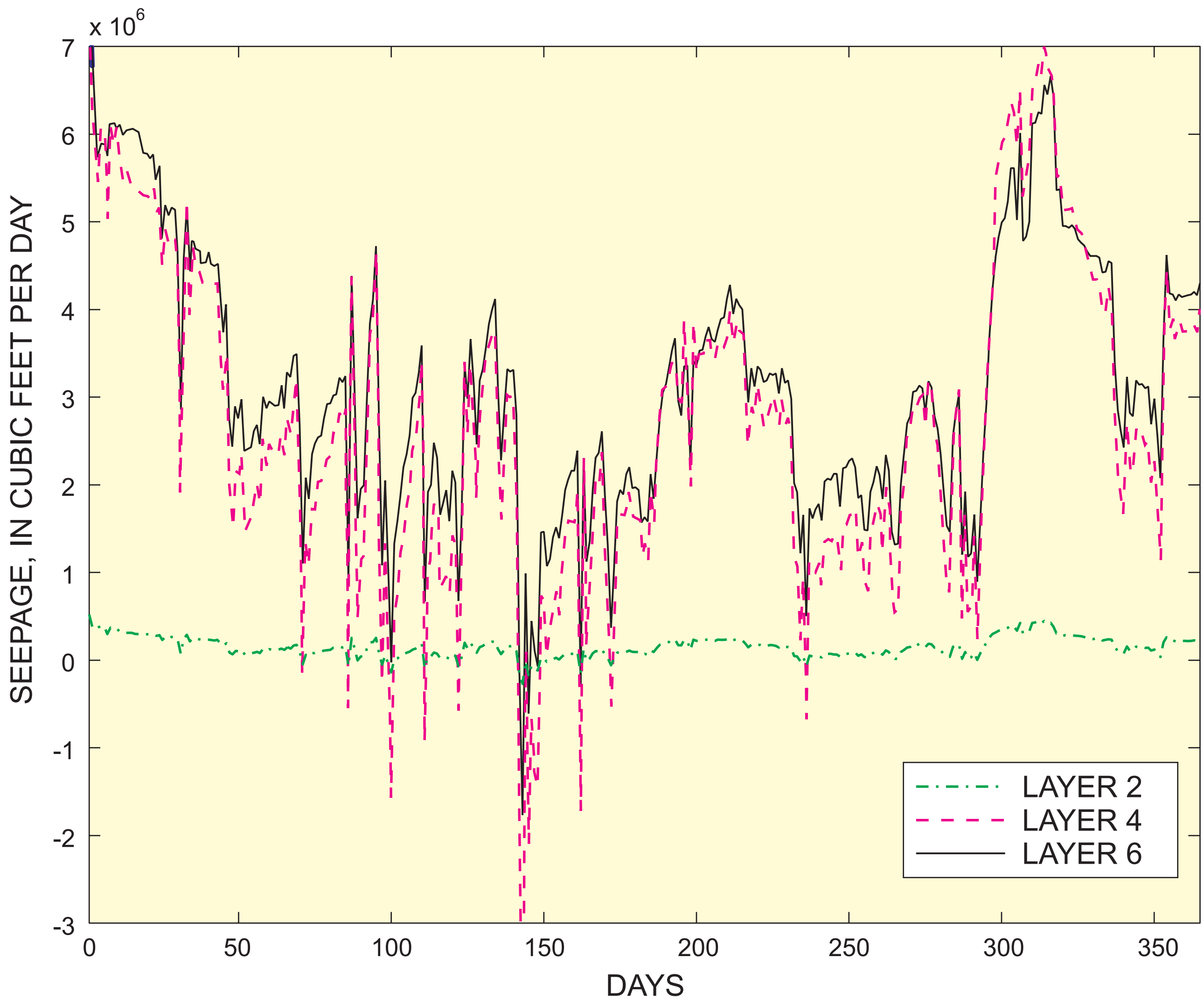


Figure 26. Seepage beneath Levee 31N by model layer using the reach-transmissivity model for 1996.

always dry, and layers 3 and 5 have such low transmissivities that seepage in them was negligible. The total flows for all layers were summed and converted to a seepage rate per foot of distance along the levee. The mean seepage rates per foot of levee were 198.9 ft^3/d in 1996 and 179.1 ft^3/d in 1997.

Head differentials for numerous combinations of pairs of monitoring stations were calculated; one station of each pair was located on the west side of Levee 31N and the other was located on the east side. A least squares linear regression analysis was then performed to fit the head differential data to the seepage rates calculated by MODBRANCH. The resulting regression equations were of the form:

$$q = C_1(H_w - H_e) + C_2 \quad , \qquad (19)$$

where q is the seepage rate per foot along the levee; H_w and H_e are the heads at the monitoring station west and east of Levee 31N, respectively; and C_1 and C_2 are coefficients obtained by the regression. Higher-order polynomial regressions also were performed, but provided no discernible improvement in fit. The standard deviation, coefficient of determination, and regression coefficients for several selected sets of monitoring station pairs are summarized in table 11; many other combinations of stations were investigated as well. The fit was poor for many of the station pairs. The pair with the best results was wells G-3577 and G-3551, followed by well G-3578 and the L-31N Canal at Mile 4. The regression equations for these two pairs were further examined with the data from 1997. The regression coefficients obtained for the 1996 data were used to predict seepage with measured heads from 1997; the performance of the regression equations was then evaluated (figs. 27 and 28). The station pair of wells G-3577 and G-3551 exhibited better correlation to MODBRANCH data in all evaluated statistical categories for both calibration and verification.

Although the algorithm based on wells G-3577 and G-3551 provides slightly better seepage estimates, the algorithm employing well G-3578 and L-31N Canal at Mile 4 may have more practical applications because one of the parameters is canal stage rather than ground-water head. Many water-management decisions in southern Florida involve gate operations. Canal stage responds quickly to gate operations and is much more subject to human control than is ground-water head. As a result, a seepage estimation algorithm including canal stage provides an operational benefit in comparison to an algorithm where ground-water heads are the only input parameters.

Table 11. Seepage algorithm statistical properties and coefficients for least squares regressions based on head differences for 1996

[Standard deviation of MODBRANCH output is 112.66 square feet per day per foot of levee. C1 and C2 are coefficients obtained by linear regression]

West station	East station	Standard deviation (square feet per day)	Coefficient of determination (R^2)	C_1 (feet per day)	C_2 (square feet per day)
G-3575	L-31N Mile 4	23.14	0.042	91.85	179.57
G-3575	G-3551	39.40	.122	681.57	45.93
G-3575	G-3553	40.73	.131	219.55	-4.39
G-3575	G-3555	52.07	.214	215.59	-75.81
G-3577	L-31N Mile 4	42.19	.140	138.03	121.41
G-3577	G-3551	77.50	.473	642.83	-170.96
G-3577	G-3553	65.11	.334	289.97	-171.37
G-3577	G-3555	68.23	.367	237.02	-186.28
G-3578	L-31N Mile 4	74.74	.440	373.06	-131.36
G-3578	G-3551	42.92	.145	161.21	53.88
G-3578	G-3553	43.30	.148	112.28	19.17
G-3578	G-3555	51.58	.210	122.48	-39.80

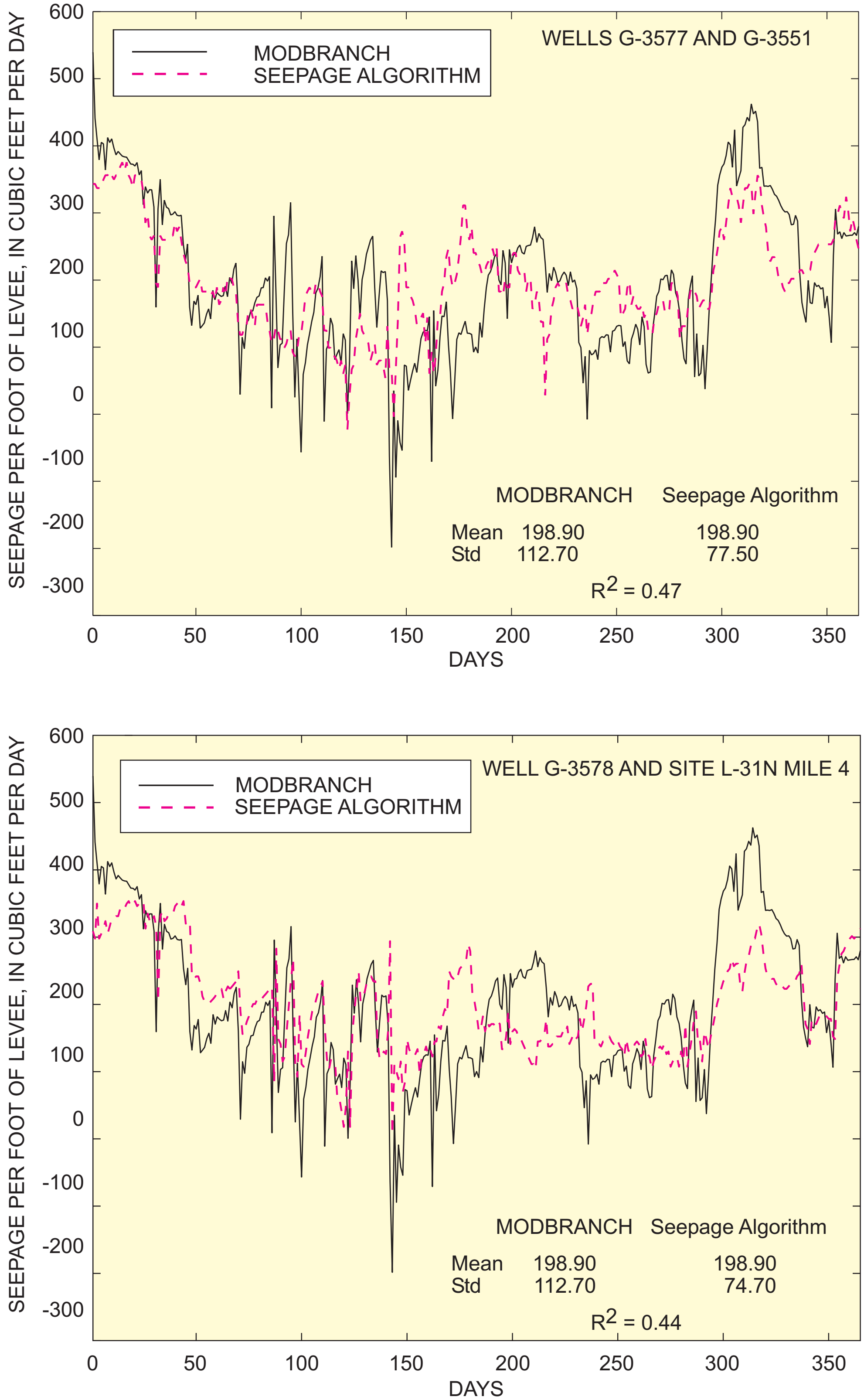


Figure 27. Seepage algorithm calibration using selected station pairs and MODBRANCH seepage output for 1996

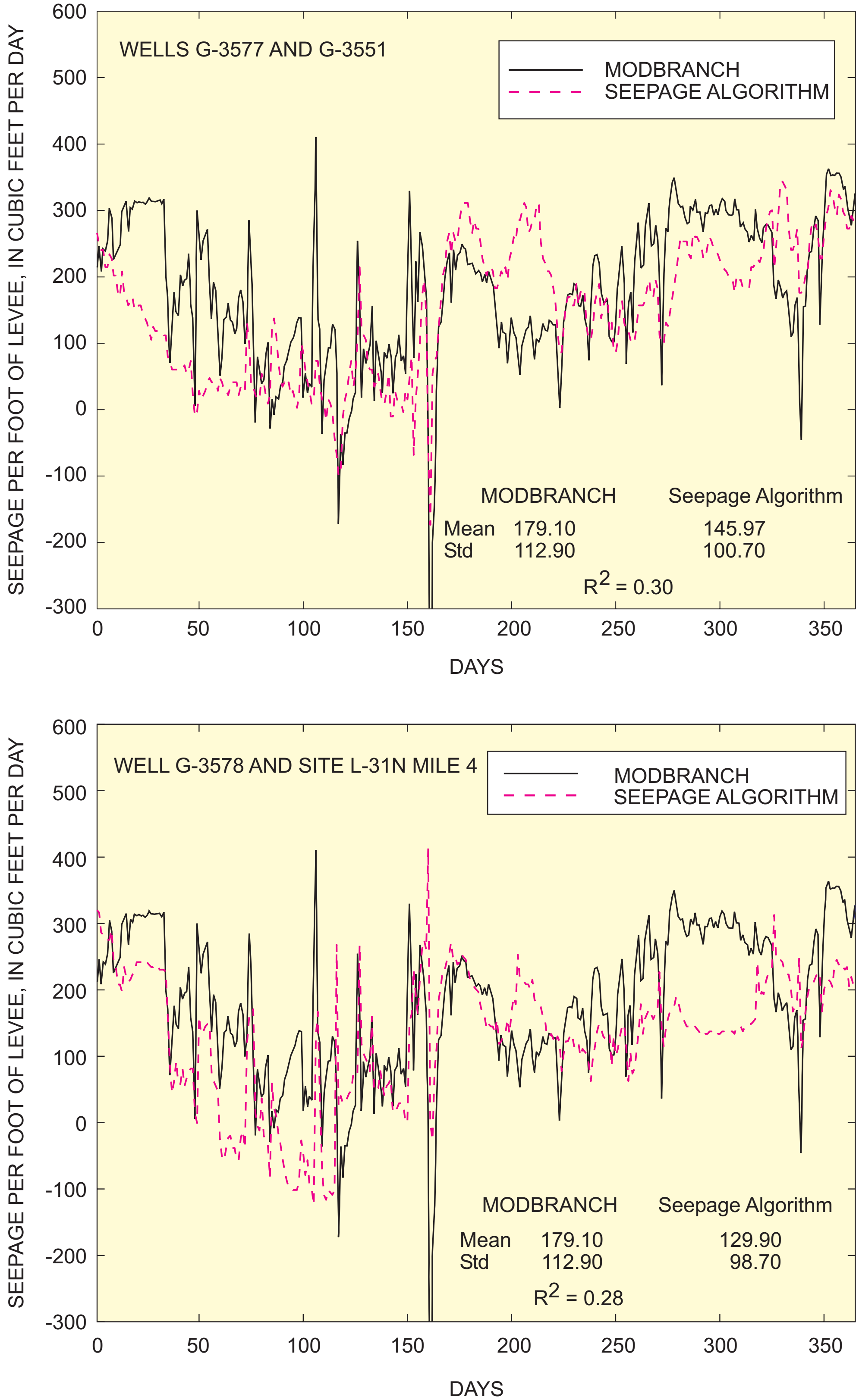


Figure 28. Seepage algorithm output using selected station pairs and MODBRANCH seepage output for 1997.

To ensure that the MODBRANCH results and the seepage algorithm were consistent with the observed hydrogeologic properties of the aquifer, a seepage calculation based directly on Darcy's law was made. The head gradient was obtained from measurements at wells G-3577 and G-3551 as in the seepage algorithm. In this calculation, Darcy's law may be stated as:

$$q = T\frac{(H_w - H_e)}{L} , \tag{20}$$

where T is the aquifer transmissivity, and L is the distance between the points where the head is measured. Flow was calculated using the calibrated transmissivity of 1,400,000 ft/d; the distance between wells G-3577 and G-3551 is 1,801 ft. Results of this transmissivity-based estimate are shown in figure 29; seepage is higher than in the results of the model and algorithm, but the differences are smaller in magnitude than the range of previous estimates of aquifer transmissivity at this site (Chin, 1990; Fish and Stewart, 1991).

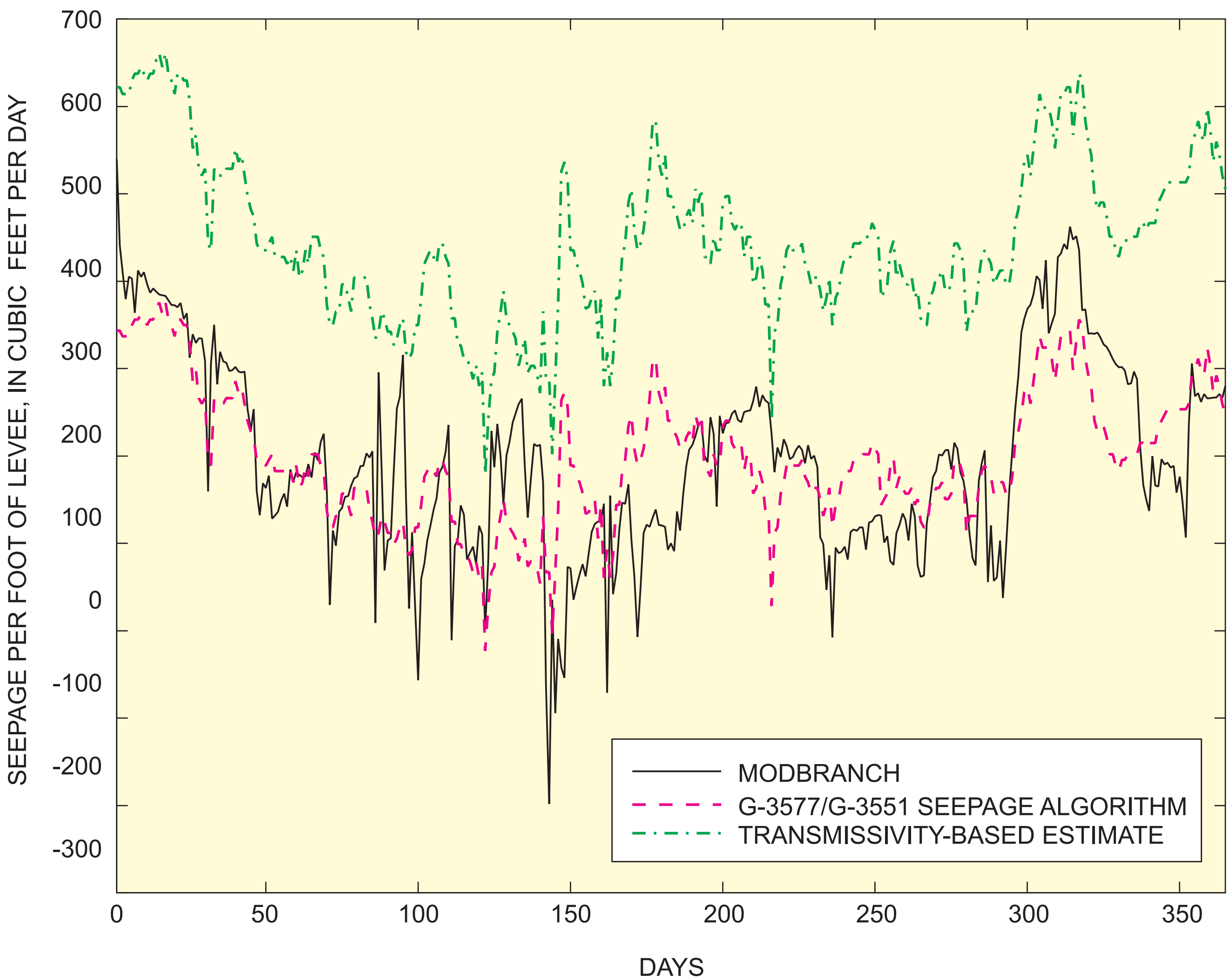


Figure 29. Levee 31N seepage estimate based on aquifer transmissivity for 1996.

The usual comparisons of measured and modeled values cannot be made for seepage rates because independent measurements of ground-water seepage velocity do not exist. Assessment of the reliability of seepage estimates consists of examining the accuracy of the model in simulating ground-water head and canal stage and discharge or by performing analytical calculations based on numerous simplifications. The development and employment of techniques to directly measure seepage rates would provide an independent basis for evaluation of model results.

SUMMARY

Levee 31N separates the wetlands of the Everglades from suburban Miami. It is believed that eastward seepage of ground water beneath the levee (out of the wetlands and toward Miami) constitutes a substantial portion of the Everglades water budget in this area. To understand the effects of water-management practices in the area, a study was conducted to quantify ground-water seepage beneath Levee 31N. Additionally, the feasibility of calculating leakage based on reach transmissivity was tested.

Vertical seepage in the wetlands was quantified by direct measurement using seepage meters. Seepage rates were observed to vary seasonally and increase with proximity to Levee 31N. Ground-water flow within the Biscayne aquifer was quantified using the MODBRANCH computer model. MODBRANCH is a finite-difference model that links a ground-water model (MODFLOW) and a surface-water model (BRANCH) by calculating leakage between surface-water channels and the ground-water aquifer based on the assumption of vertical flow through a low-permeability layer at the bottom of the channel.

The computer model developed for this study encompassed a region centered on Levee 31N that included both wetland areas of the Everglades and non-wetland areas of western Miami. The model grid consisted of 49 rows, 58 columns, and 6 layers. Row and column spacing was 500 ft near the center of the study area and 1,000 ft elsewhere. The top layer was assigned a hydraulic conductivity of 3,000,000 ft/d to simulate the wetlands environment; the hydraulic conductivities of the other layers were based on geologic properties of the surficial aquifer, which is exceptionally transmissive. Each MODFLOW stress period was 1 day, and each BRANCH stress period was 1 hour. The model was run to simulate transient conditions throughout a calendar year. Ground-water boundary conditions consisted of interpolated heads obtained from monitoring wells and canal gaging stations around the perimeter of the study area; initial conditions were obtained by using an inverse distance method to interpolate measured heads at the beginning of the simulation. Evapotranspiration and recharge were obtained from measured data. Potential evapotranspiration was constant throughout the model area, and the extinction depth was specified as 20 ft, based on previous research. Recharge values, which were obtained from three rain gages within the study site, were spatially variable. A 6-mi reach of the L-31N Canal was simulated by BRANCH; boundary conditions consisted of specified head and discharge at the upstream and downstream ends of the channel, respectively.

Differences between the existing method (vertical-flow relation) used in MODBRANCH to calculate leakage between the ground water and surface water and an alternate method (reach-transmissivity relation) were examined. The input parameters required for the reach-transmissivity relation have more of a physical basis, and are usually easier to obtain from published sources than those required for the vertical-flow relation. The reach-transmissivity parameters also are less dependent on model-grid spacing. The equations associated with the reach-transmissivity relation were developed in finite-difference form and incorporated into a modified version of MODBRANCH. The program was then tested for three problems with analytical solutions and with one problem that had previously been solved using the original version of MODBRANCH. The reach-transmissivity relation was judged to have functioned satisfactorily in these tests.

The computer model of the Levee 31N area was first run using the existing version of MODBRANCH (with the vertical-flow relation) and calibrated by varying aquifer hydraulic conductivity and the vertical-flow leakage coefficient. Calibration was based on data from the 1996 calendar year. The model was found to be more sensitive to changes in the aquifer hydraulic conductivity than in the leakage coefficient. The overall transmissivity of the Biscayne aquifer was calibrated to 1,400,000 ft/d, and the vertical-flow leakage coefficient was established as 0.0009 s^{-1}; both results were similar to those of previous studies.

The version of MODBRANCH modified to use the reach-transmissivity relation was calibrated with the aquifer hydraulic conductivities previously obtained using the vertical-flow relation. When aquifer transmissivity is fixed, the reach-transmissivity leakage coefficients are only a function of the distance from the channel where the reference ground-water head is obtained. The best results were obtained when this distance was such that the ground-water head was obtained from the model cells directly adjacent to the channel; results were similar for varying distances. There were no large differences between results modeled using the vertical-flow and reach-transmissivity leakage relations. The mean annual modeled ground-water heads differed by only 0.02 ft, and the mean yearly modeled canal discharge varied less than 1.0 ft^3/s. The output of vertical-flow and reach-transmissivity models provided R^2 values within 0.03 of one another for ground-water head, canal stage, and canal discharge. The reach-transmissivity version of MODBRANCH, however, reached a solution in about 40 percent fewer iterations.

Comparison of seepage rates determined by the seepage meters in the wetlands west of Levee 31N and those determined by the MODBRANCH model revealed important differences. These differences were believed to be a result of local nonuniformity in aquifer properties.

An estimation of seepage beneath Levee 31N was obtained by summing the MODFLOW cell-by-cell flow terms of the model cells directly west of the levee. From these data, an algorithm to estimate seepage beneath Levee 31N was developed by linear regression; the best results were obtained by calculating seepage as a function of the head difference between wells G-3551 and G-3577. The development of the seepage algorithm was based on data from the 1996 calendar year; data from the 1997 calendar year were then used for verification. The algorithm was determined to adequately predict ground-water seepage on the basis of comparisons to the MODFLOW results.

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