# AN OPERATIONAL WATERSHED MODEL: STEP 1-B; REGULATION OF WATER LEVELS IN THE KISSIMMEE RIVER BASIN

by

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ABSTRACT

Step-1B in the first phase of an "Operational Watershed Model" recently initiated by the Central and Southern Florida Flood Control District for managing its water resources is to compute water surface elevation, spatially and temporally, in the Kissimmee River Basin system of reservoirs, channels and spillways. An approach is based upon the principles of gradually varied flow. Mathematical relationships to compute lake stage as a function of storage, and to compute discharge through the control structures as a function of gate opening and differential head across the structures are developed. Their feasibility for application is clearly demonstrated by the simulated mean daily water surface elevation for a period of two years on the tail side of one typical gated spillway and on the head side of another gated spillway.

SYMBOLS:

A = Cross-sectional area of the channel

a = Constant

B = Bed elevation from mean sea level at point 1

b = Constant

BF = Balancing factor

C = Bed elevation from mean sea level at point 2

CONST = A factor to convert cubic feet per second into acre-feet

D = Absolute difference between simulated and recorded stage

E = Evaporation from lake surface

EH = Effective head

FLL = Unmeasured local inflow into a lake

GO = Effective gate opening

g = Acceleration due to gravity

HR = Hydraulic radius

I = Inflow into a lake

LOCK = Amount of water flowing out of a lake when the lock is opened for a boat to cross the structure

N = Structure number

O = Outflow from a lake

P = Wetted perimeter

p = Constant

Q = Discharge through a structure

$R^2$ = Coefficient of determination

r = Constant

RN = Manning's roughness coefficient

S = Storage

s = Constant

SE = Energy gradient

SEEP = Seepage through the structure and otherwise

SO = Slope along the stream bed

T = Top width of the channel cross-section

t = Time

V = Velocity of flow

WSE = Water surface elevation

x = Horizontal distance between two points along the channel bed

Y = Maximum limit on gate opening or depth of water at the weir crest

$y$ = Stream depth

$z$ = Absolute difference in bed elevation of two points along the channel

$\Delta S$ = Change in lake storage

$\alpha$ = Velocity head coefficient

## INTRODUCTION

The Central and Southern Florida Flood Control District has initiated a program for development of an "Operational Watershed Model" of the District system. Since system planning and design has been largely completed, and major elements of the physical system are now in existence, the model development program is operations-oriented. It is expected that such a model, when fully developed and tested, will be a valuable tool for use in determining operating procedures and in guiding management of the water resource.

Selected for initial development was a model of a portion of the District system; the Kissimmee River Basin. The first phase, divided into Step-1A and Step-1B, is regulation of water levels in the Kissimmee River Basin. However, initial development in this phase is a mathematical simulation of the physical system which would afford a thorough understanding of the factors governing basin behavior.

Step-1A is expected, in general, to transform an input (rainfall) to the system (Kissimmee River Basin) into an output (streamflow). More specifically, Step-1A will produce a time-water release curve at every control structure in the basin. The term water release in this paper is synonymous to the term streamflow.

Step-1B is expected to answer questions concerning the state of the physical system of reservoirs, channels and spillways under dynamic conditions. That is; with a given set of inputs, such as the output from Step-1A, and spillway gate operations, what will be, at any time, the amount of water in storage and the streamflow conditions in the system? The converse of this question is, of course, pertinent also: with the inputs from Step-1A and some desired set of conditions of system storage and streamflows, what will be the required operation at each decision-making point (gated spillway structure)? Such questions must be answered in order to develop rational procedures for both flood control and water supply operations. In the first case an answer is required in order to route flood flows through the system without exceeding safe storage levels in the reservoirs and without reaching damaging flow rates and velocities in the channels and at the spillways. In the second case an answer is required in order to manipulate and adjust allocations of available water for various uses both spatially and temporally.

Answers, in general, to the questions posed above could be provided by information, corresponding to the given set of inputs, on variations in water surface elevation within the system with space and time. In particular, a knowledge of head-and-tail-water elevation at any time at every control structure in the river basin system should serve the purpose. Thus, the final outcome of the first phase will be of the nature presented in Figure 1. Figure 1(a) is an output from

Step-1A, and Figure 1(b) and 1(c) are outputs from Step-1B. The curves presented in Figure 1 are arbitrary.

It is clear then, that the first phase of this operational watershed model is aimed toward the regulation of water levels in the Kissimmee River Basin. Regulation of water levels in lakes has been based by several workers: Megerian and Pentland (1968), Clark and Cavadias (1967), Fiering (1964), and Granger (1964), upon such statistical methods as multivariate technique and time series analysis. Having these as a background, this paper is intended to present a basic approach under Step-1B and its feasibility for simulating water surface elevation at the tail-and-head side of control structures in the Kissimmee River Basin.

## GENERAL

The Kissimmee River Basin, Figure 2, extends over approximately 3,000 square miles of the District's total area of 16,000 sq. miles. Present efforts are being concentrated to model the upper chain (area above Structure 65) of the basin. In the upper chain, nine control structures are in operation and about sixteen more are being planned to channelize the flow of water. All the control structures in operation, except two, are equipped with vertical lift gates over ogee weirs and a device to record head-and-tail water elevation. Each of these two exceptions have culverts with two corrugated metal pipes instead of ogee weirs. Control Structures 61 and 65 are equipped with locks for navigation purposes. The flow of water through the structures is always under submerged condition.

## A BASIC APPROACH

The approach consisted essentially of two parts. The first part was a qualitative delineation of the basin into typical systems. The second part was the development of mathematical relationships that would enable the determination of WSE, spatially and temporally, within the basin.

Qualitative delineation of the basin into typical systems. It is clear from Figure 2 that three typical systems exist, or are expected to exist in the upper chain of the Kissimmee River Basin. These three typical systems are presented in Figure 3. In system 1, water enters through a canal. Flow into the lake through a canal is controlled by a structure. Outflow from the system into a canal is controlled just at the outlet of the lake by another structure. In system 2, water enters through a canal. Flow into the lake through a canal is controlled by a structure. Water flows from the lake into another canal. Outflow from the system is controlled by another structure. In system 3, water enters through a canal. Flow into another canal instead of a lake is controlled by a structure. Outflow from the system into a canal is controlled by another structure. Such a delineation would be helpful in developing a computer programming mechanism for the whole basin system.

Mathematical relationships. It is readily seen from Figure 3 that the tailwater elevation (TWE) at any structure in the three systems can be obtained by computing a water surface profile upstream. The headwater elevation (HWE) in system 1 can be assumed equal to WSE of the lake and the HWE in system 2 and system 3 can be obtained by computing a water surface profile downstream. Accordingly, the computation of water surface elevation at the control structure was based upon the principles of unsteady or gradually varied flow. Garrison, et al (1969) have used a numerical method developed by Stoker (1953-54) to simulate unsteady flow conditions which have occurred or which are expected to occur in some of TVA's system of reservoirs and natural river channels. Prasad (1968) has used the differential equation for gradually varied flow in a numerical technique, called the trapezoidal rule, to compute the water surface profile along a stream channel. Several other methods of flow profile computation are those given by Chow (1955), Keifer and Chu (1955), and Pickard (1963). A change, however, in water surface elevation (WSE) at any time with space, as seen from Figure 4, can basically be represented by an equation.

$$\frac{d(WSE)}{dx} = \frac{dB}{dx} + \frac{dy}{dx}, \quad B = C + z \tag{1}$$

Integrating equation 1 we get

$$WSE = B + y = C + z + y \tag{2}$$

where WSE = water surface elevation,

B = bed elevation from mean sea level at point 1,

C = bed elevation from mean sea level at point 2,

z = (SO) multiplied by x, and

y = depth of water.

The differential equation to determine a change in depth of water with space, $^{dy}/_{dx}$, for gradually varied flow, is

$$\frac{dy}{dx} = \frac{SO - SE}{1 - \frac{\alpha Q^2 T}{gA^3}} \tag{3}$$

where $y$ = depth of water or stream depth,

$x$ = distance along the channel bed,

SO = slope along the stream bed,

SE = energy gradient,

$\alpha$ = velocity head coefficient,

Q = discharge through the control structure,

T = top width of the channel cross-section,

g = acceleration due to gravity, and

A = cross-sectional area of the channel.

In equation 3,

$$SE = \frac{(RN)^2\ V^2}{2.22(HR)^{4/3}} = \frac{(RN)^2\ Q^2\ P^{4/3}}{2.22A^{10/3}}\ ,\ V = \frac{Q}{A}\ ,HR = \frac{A}{P}$$

where V = velocity of flow,

RN = Manning's roughness coefficient,

HR = hydraulic radius, and

P = wetted perimeter.

A detailed argument about the development of equation 3 is given by Chow (1959).

The discharge, Q, through a structure in equation 3 is determined by a relationship of the form presented below.

$$Q(N) = p(GO)^r(EH)^s,\ 0 \leq GO \leq Y,\ EH \geq 0 \qquad (4)$$

where N = structure number,

GO = effective gate opening,

EH = effective head, i.e., difference in head across the structure,

Y = depth of water at the weir crest or maximum limit on gate opening, and

p, r, and s = constants.

A technique to determine WSE of the lake at any time is based upon the following equation:

$$(ST)_{t+1} = (ST)_t + (\Delta S)_{t+1} \tag{5}$$

where ST = WSE of the lake, t = time, and $\Delta S$, the change in storage, is given by

$$\Delta S = I - O \tag{6}$$

where I is inflow into, and O is outflow from the lake.

Storage, S, is converted into WSE by a relationship of the form

$$WSE = a(S)^b \tag{7}$$

where a and b are constants.

## A CASE STUDY

Due to unavailability of required data over the entire Kissimmee River Basin, a case, Figure 5, which is an example of system 1, was selected in the upper chain to test the feasibility of the basic approach presented above. However, the parameters involved must be determined, equation 3 must be solved, and $\Delta S$ must be estimated before this approach can be tested.

Determination of p, r, and s of equation 4. Measurements of discharge, which were associated with observations on gate openings and effective head, were available for structures S-59 and S-61. The coefficients p, r, and s for both the structures were then determined by employing the least squares technique to the logarithmically transformed data. The discharge equations thus obtained for Structures 59 and 61, and the values of such important statistics as the coefficient of determination, the mean squared error, and the standard error of estimates are presented in Table 1.

Determination of a and b of equation 7. WSE-Storage curve for Lake Tohopekaliga was available. The least squares fit to this curve was obtained in the same manner as above. The WSE-Storage equation and its associated coefficient of determination, the mean squared error, and the standard error of estimates are also presented in Table 1 for Lake Tohopekaliga.

Solution to equation 3. The solution to equation 3 was obtained by an iterative procedure which used an equation given by Prasad (1968) in the form

$$y_{i+1} = y_i + \frac{\dot{y}_{i+1} + \dot{y}_i}{2} DX \qquad (8)$$

where $y_i$ = depth of water at the $i^{th}$ position along the channel bed,

$\dot{y} = {}^{dy}/_{dx}$, and it is assumed to be negative when computation proceeds upstream and positive when computation proceeds downstream, and

DX = horizontal distance between $i^{th}$ and $(i+1)^{th}$ position along the channel bed.

Equation 8 is based essentially upon a numerical integration technique called the trapezoidal rule. The details about trapezoidal rule in general is available in any numerical analysis textbook, e.g., the one written by Hildebrand (1956).

<u>Determination of SO and RN.</u> The true values of such parameters as slope along the stream bed (SO) and Manning's roughness coefficient (RN) are difficult to estimate, but are fairly easy to adjust so that simulation will reproduce the observed water surface elevation at a given point in space and time. In order to be able to adjust the values of SO and RN efficiently it is necessary to know how they separately affect the values of WSE at a point in space and time when a constant set of input is used.

The slope along the stream bed was computed as

$$SO = \frac{B-C}{x}$$

where B = bed elevation at the tail side of Structure 59, C = bed elevation at the outlet of Canal 31 into Lake Tohopekaliga, and x = horizontal distance between points B and C. Since the values of B, C, and x were to be obtained from the "As Built" drawing of Canal 31 and Structure 59, different persons were asked to compute the value of SO. These values of SO seemed to fall in an interval of $0.000141 \leq SO \leq 0.000165$. The difference in the values of SO obtained by different persons was primarily due to the differences in the values of B and C read by them. Therefore, the effects of variations in SO on WSE were studied by varying the value of SO in two ways, Figure 6.

One way was to hold the value of C constant and vary the value of B. The other way was to hold the value of B constant and vary the value of C. The results are plotted in Figures 7 and 8. The effect of an intermediate value of SO = 0.000153 on WSE has not been plotted in Figures 7 and 8 simply because it will make them congested. However, a thorough look at the computer output indicated that the curve obtained by using SO = 0.000153 lay in between the two curves shown in Figures 7 and 8. When the gate is open, Figure 7(a) shows that WSE increases with an increase in the value of SO which is opposite to that seen in Figure 7(b). Figures 8(a) and 8(b) show that at any value of SO, the values of WSE at a point in space and time could be increased and decreased by increasing and decreasing the bed elevations by the same amount at the up-and-down-stream points of the reach. When the gate is closed, no change in the values of WSE occurs with the change in the values of SO, B, or C. It appears, however, from Figures 7 and 8 that a small change in the values of SO, B, and C may not cause a big change in the values of WSE. Therefore, it was decided that for further investigations, the case presented in Figure 6(a) should be used only.

The design roughness value of the canals in the District's system was 0.03. It is now felt, however, that this value should have been much lower than 0.03; therefore, effects of three RN values, 0.025, 0.020, and 0.015 on WSE were studied by using the same set of inputs as before. The results are presented in Figure 9. Evidently, a small increase in the value of RN is associated with a large increase in the values of WSE when the gate is open. The values of WSE do not change with a change in the value of RN when the gate is closed. This is logical as well as obvious.

Since it is difficult to adjust the values of SO and RN individually for every simulation, it was decided to arrive at the value of

RN in combination with SO for Canal 31 downstream of Structure 59. For this purpose a sensitivity analysis was conducted by simulating the mean daily tailwater elevations (MTWE) at Structure 59 for the year 1967 by using various combinations of RN and SO values. The number of days for which various combinations of SO and RN produced simulated values within one-tenth of a foot difference with the observed values of MTWE are presented in Table 2. It is clear from Table 2 that there is an envelope of three RN values (0.018, 0.017, 0.016) and three SO values (0.000165, 0.000153, 0.000141) within which the use of any combination of RN and SO values may produce a near maximum or maximum number of days in a year for which simulated MTWE values at Structure 59 are within one-tenth of a foot difference with the observed values. Therefore, it was decided to select from this envelope one combination of RN and SO values that can be used in the operational watershed model for Canal 31 downstream of Structure 59. The MTWE values at Structure 59 were, therefore, simulated for one more year, 1965, by using the different combinations of RN and SO values within the envelope. The number of days in the years 1965 and 1967 for which different combinations of RN and SO values in the envelope produced simulated values within one-tenth of a foot difference with the observed values of MTWE were added together. These values are plotted in Figure 10. Using the maximum number of days (sum of the days in the years 1965 and 1967 for which simulated values were within one-tenth of a foot difference with observed MTWE values) as a criterion, the values of RN and SO selected for Canal 31 downstream of Structure 59 are respectively 0.018 and 0.000165.

RAIN = amount of rainfall on the lake surface. It was assumed negligible.

Simulation. Having determined the values of the parameters involved, a method to solve the differential equation 3, and an expression to determine the $\Delta S$ in Lake Tohopekaliga, the feasibility of this approach was tested by simulating the mean daily tailwater elevation (MTWE) at Structure 59 and mean daily headwater elevation (MHWE) at Structure 61 for the years 1965 and 1966. The results are presented in Figures 11, 12 and 13. Also, the distribution of the magnitude of absolute difference between simulated and recorded MTWE and MHWE values by number of days in a year are shown in Table 3.

Most of the simulated MTWE values are within one-tenth of a foot difference with the observed values. A few of them are between one-tenth and fifteen-hundredth difference, while a very few are beyond this difference. Since there are only 270 days in the year 1966 for which simulated values are within one-tenth of a foot difference with the observed MTWE values, simulation was conducted by using other combinations of RN and SO values in the envelope. The results seemed to indicate that the combination of RN and SO values selected by sensitivity analysis for Canal 31 downstream of Structure 59 may be appropriate.

Most of the simulated MHWE values are within one-tenth of a foot difference with the observed values. A considerable number of values are between one-tenth and two-tenths of a foot difference, while some of the values are beyond this difference. Efforts were not made to improve the estimate of $\Delta S$ in Lake Tohopekaliga because a very good estimate of change in lake storages could be obtained with availability of output from Step-1A as an input to Step-1B.

## CONCLUSION AND REMARKS

A very high $R^2$ value, a very low mean squared error, and very low standard error of estimates associated with each equation in Table 1 indicate that the mathematical relationships proposed as equations 4 and 7 are satisfactory. A small change in the values of SO, B, C, and RN may not cause a big change in the simulated results, but there exists a very narrow interval on the combination of RN and SO values in which any combination of RN and SO values would produce a near maximum or maximum number of days in a year within one-tenth of a foot difference with observed MTWE values at Structure 59. Within the range of the data studied, the values of SO and RN established by sensitivity analysis for Canal 31 downstream of Structure 59 seem appropriate. These values of SO and RN are 0.000165 and 0.018, respectively. Over and above the results of simulation in this case study, particularly the mean daily tailwater elevations for two years period at Structure 59, clearly demonstrate the feasibility of this approach. Consequently, it has been decided to extend this approach over the whole upper chain of the Kissimmee River Basin and collection of necessary data for this purpose is in progress. Since a good estimate of $\Delta S$ could be obtained when output from Step-1A will be available as an input to Step-1B, efforts were not made to improve the simulated MHWE values at Structure 61. It is hoped that by early 1970, when both the Steps are combined, answers will be available to the questions posed in the introduction section of this paper.

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Table 1. Discharge and WSE-Storage equations for Structure 59 and Structure 61 and Lake Tohopekaliga in Figure 5

| Equations | Coefficient of determination $R^2$ | Mean Squared Error | Standard error of estimates r | s | b |
|---|---|---|---|---|---|
| $Q(59)=125.21(GO)^{1.1}(EH)^{0.255}$ | 0.992 | 0.069 | 0.042 | 0.067 | |
| $Q(61)=122.6(GO)^{1.142}(EH)^{0.519}$ | 0.986 | 0.127 | 0.034 | 0.055 | |
| $WSE=16.5935(S)^{0.1015}$ | 0.978 | 0.011 | | | 0.003 |

Table 2. Number of days in the year 1967 for which simulated values of MTWE at Structure 59 are within one-tenth of a foot difference with observed values

| Manning's roughness RN | Slope along the channel bed, SO 0.000165 | 0.000153 | 0.000141 |
|---|---|---|---|
| 0.025 | 187 | 186 | 188 |
| 0.024 | 188 | 188 | 191 |
| 0.023 | 197 | 198 | 202 |
| 0.022 | 203 | 207 | 212 |
| 0.021 | 222 | 221 | 224 |
| 0.020 | 244 | 239 | 261 |
| 0.019 | 274 | 286 | 296 |
| 0.018 | 312 | 320 | 324 |
| 0.017 | 333 | 325 | 318 |
| 0.016 | 322 | 313 | 292 |
| 0.015 | 292 | 284 | 277 |

Table 3. Distribution of magnitudes of absolute difference between simulated and recorded MTWE and MHWE by number of days in a year

| Absolute difference D (ft.) | Number of days in a year: mean daily tailwater elevation MTWE | | mean daily headwater elevation MHWE | |
|---|---|---|---|---|
| | 1965 | 1966 | 1965 | 1966* |
| $.10 \geq D \geq 0$ | 309 | 270 | 201 | 232 |
| $.15 \geq D > .10$ | 47 | 58 | 57 | 64 |
| $.20 \geq D > .15$ | 5 | 32 | 58 | 23 |
| $.25 \geq D > .20$ | 3 | 5 | 29 | 8 |
| $.30 \geq D > .25$ | 1 | 0 | 14 | 12 |
| $.35 \geq D > .30$ | 0 | 0 | 5 | 1 |
| $.40 \geq D > .35$ | 0 | 0 | 0 | 1 |
| $.45 \geq D > .40$ | 0 | 0 | 1 | 0 |

*24 days of gate operations data (October 1 through October 24) at Structure 61 was missing, so the simulation was done for only 341 days.

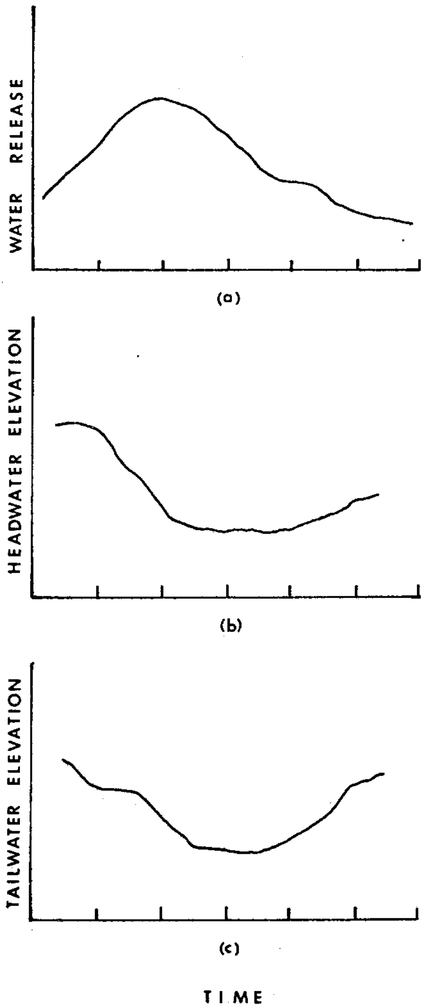


FIGURE 1. INFORMATION WANTED AT EVERY CONTROL STRUCTURE

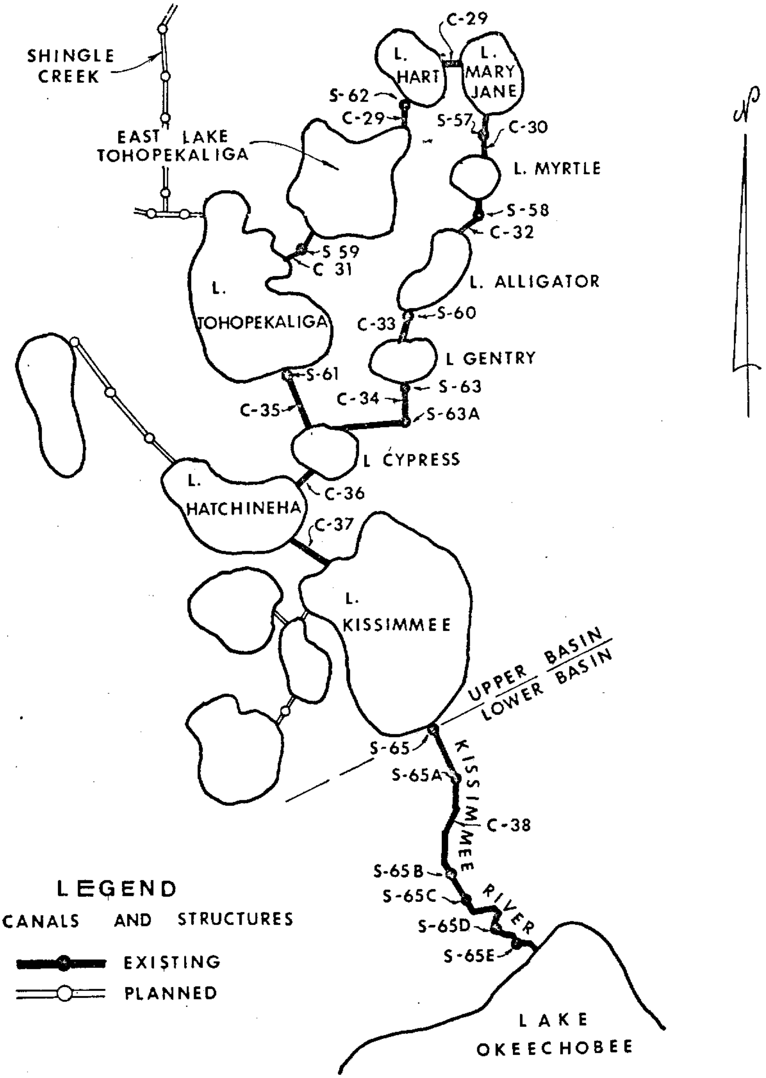


FIGURE 2 KISSIMMEE RIVER BASIN

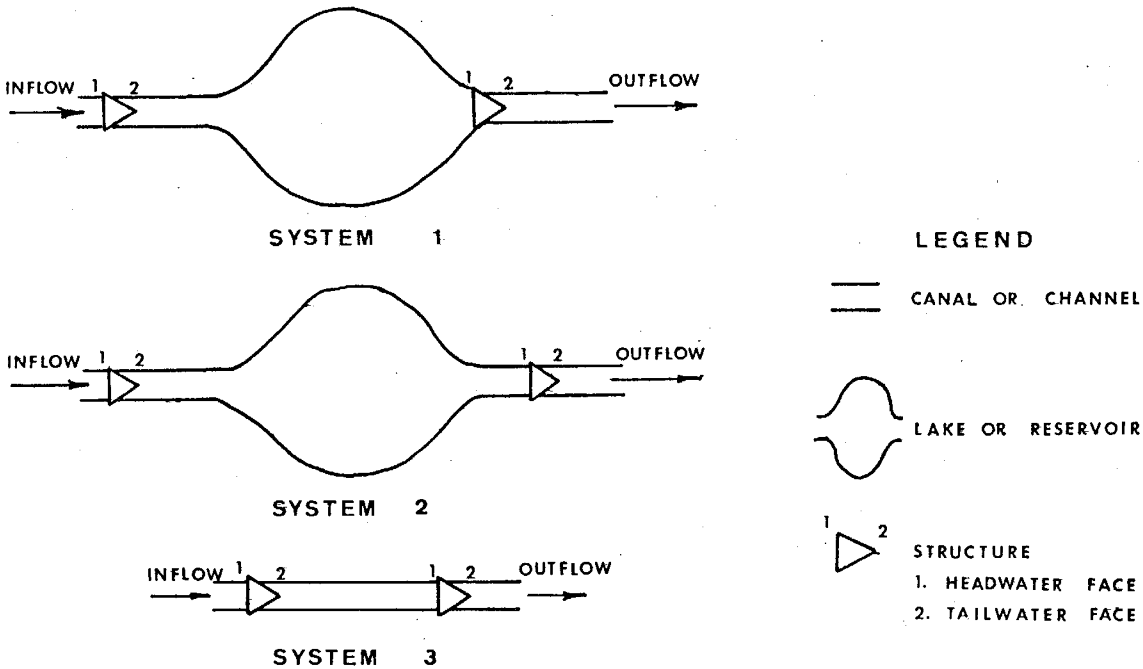


FIGURE 3. TYPICAL SYSTEMS THAT EXIST OR ARE EXPECTED TO EXIST IN THE UPPER CHAIN OF THE KISSIMMEE RIVER BASIN

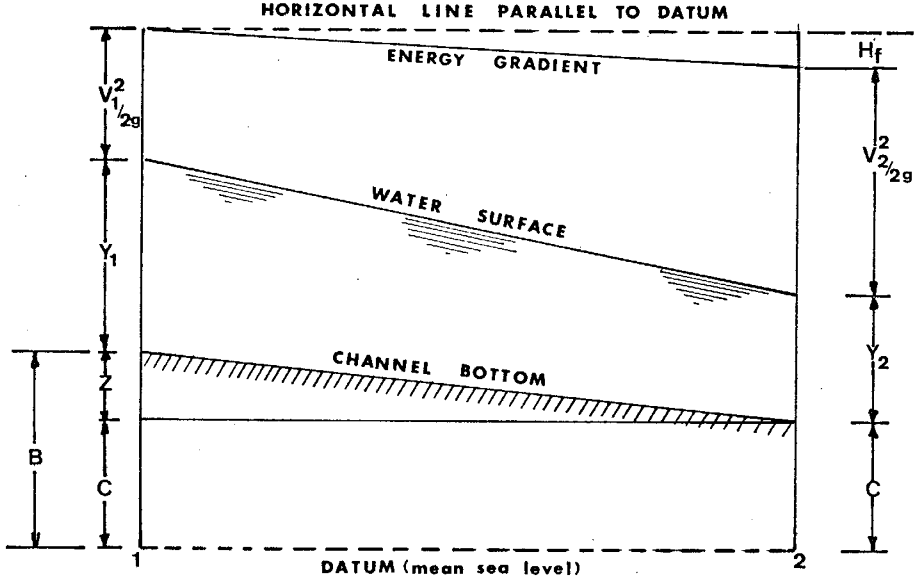


FIGURE 4. DEFINITION SKETCH FOR NON-UNIFORM FLOW IN AN OPEN CHANNEL

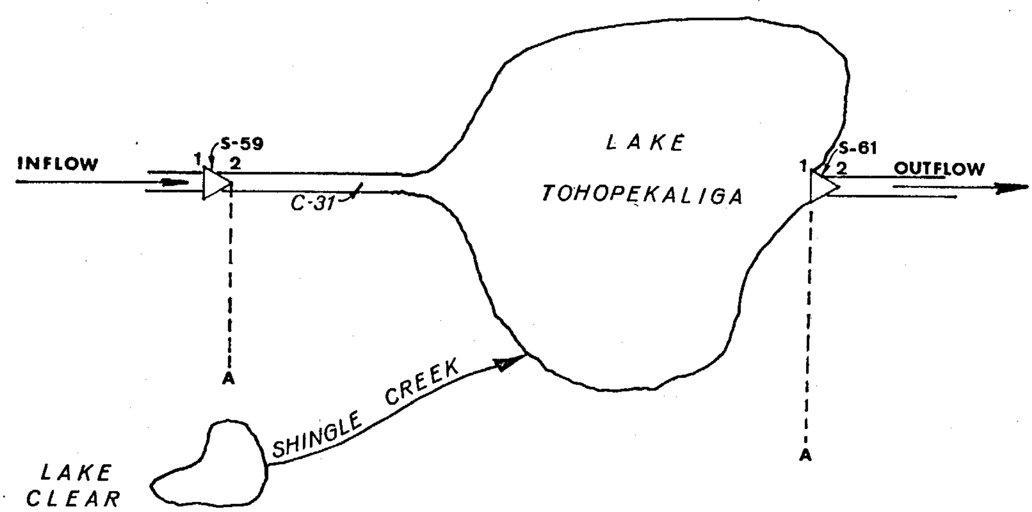


FIGURE 5. A CASE SELECTED(A-A) FOR STUDY IN UPPER CHAIN OF KISSIMMEE RIVER BASIN

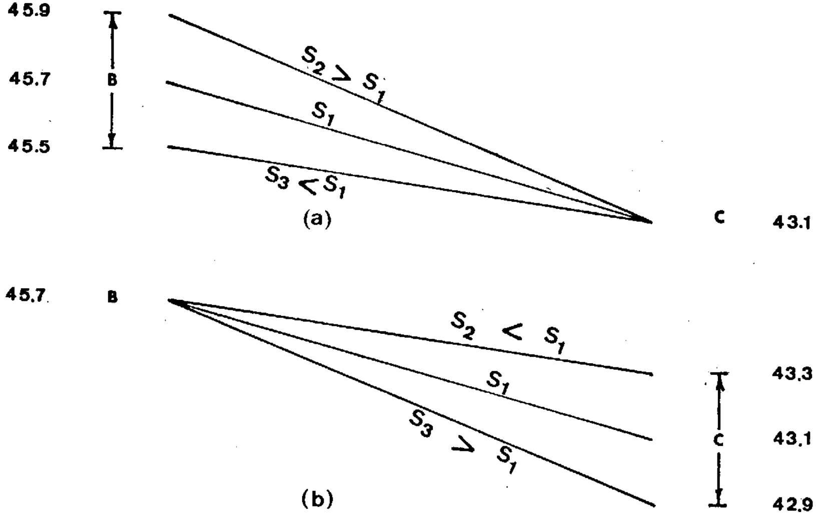


FIGURE 6. COMPUTATION OF SLOPE ALONG THE STREAM BED, SO

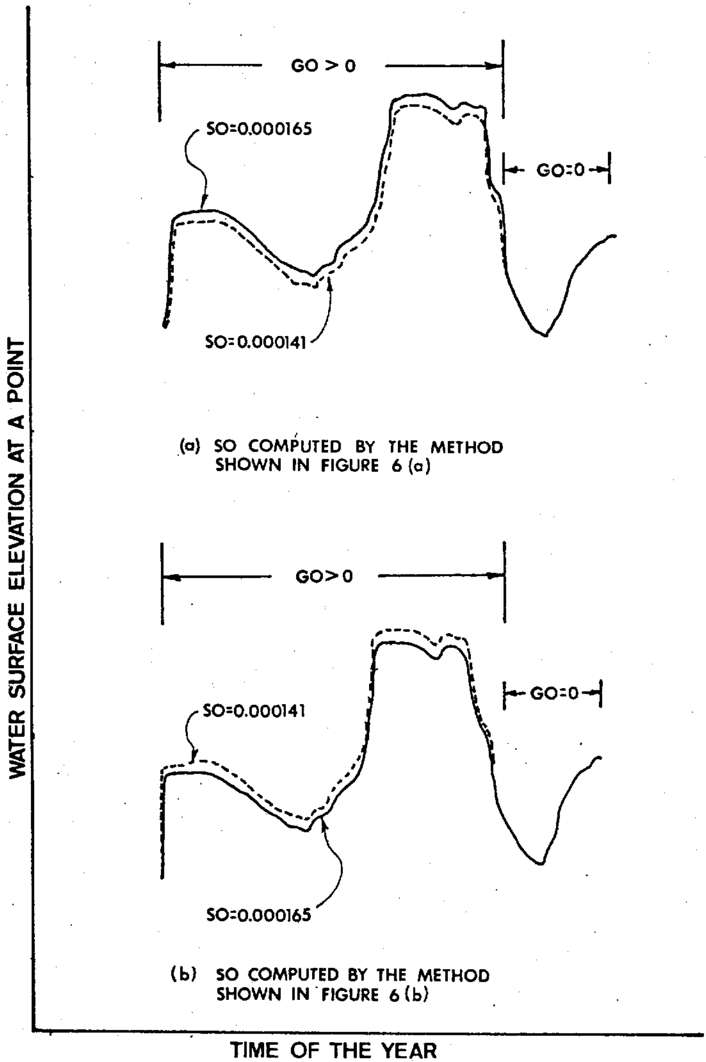


FIGURE 7. EFFECT OF SLOPE ALONG THE STREAM BED (SO) ON WATER SURFACE ELEVATION AT A POINT

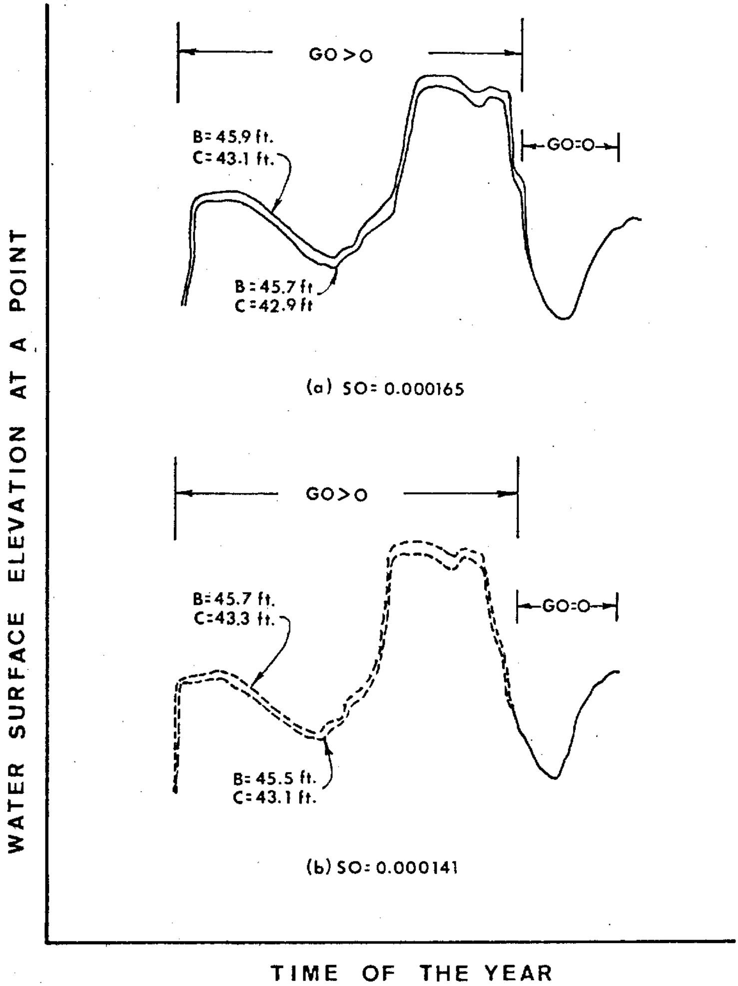


FIGURE 8. EFFECT OF BED ELEVATION (B and C) ON WATER SURFACE ELEVATION AT A POINT

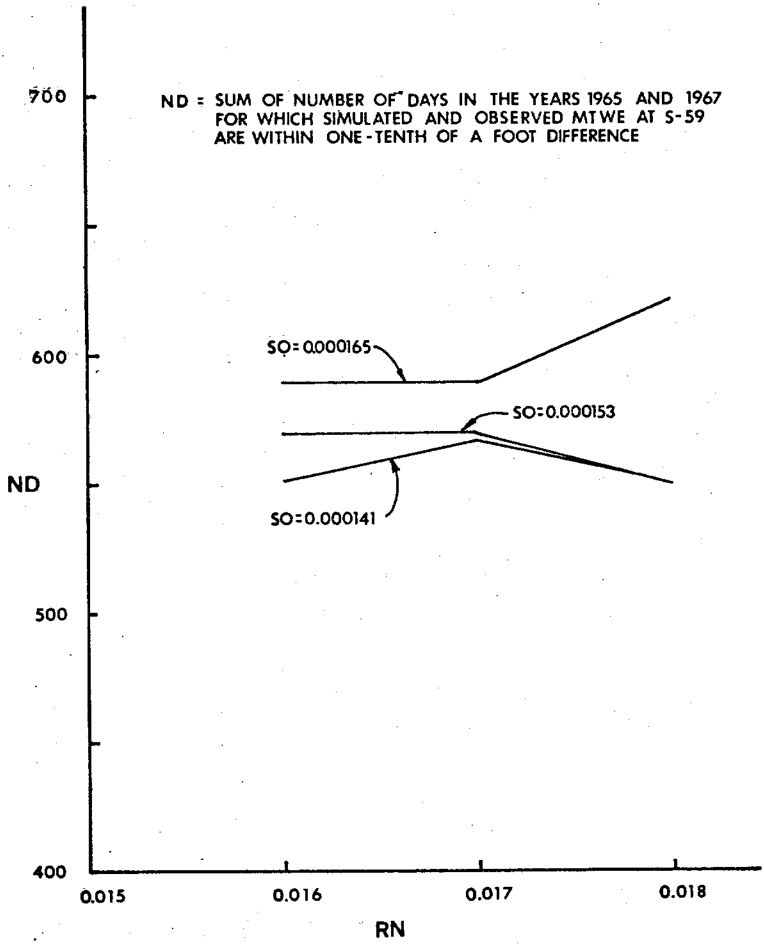


FIGURE 10. EFFECT OF THE SELECTED COMBINATIONS OF RN AND SO VALUES ON ND

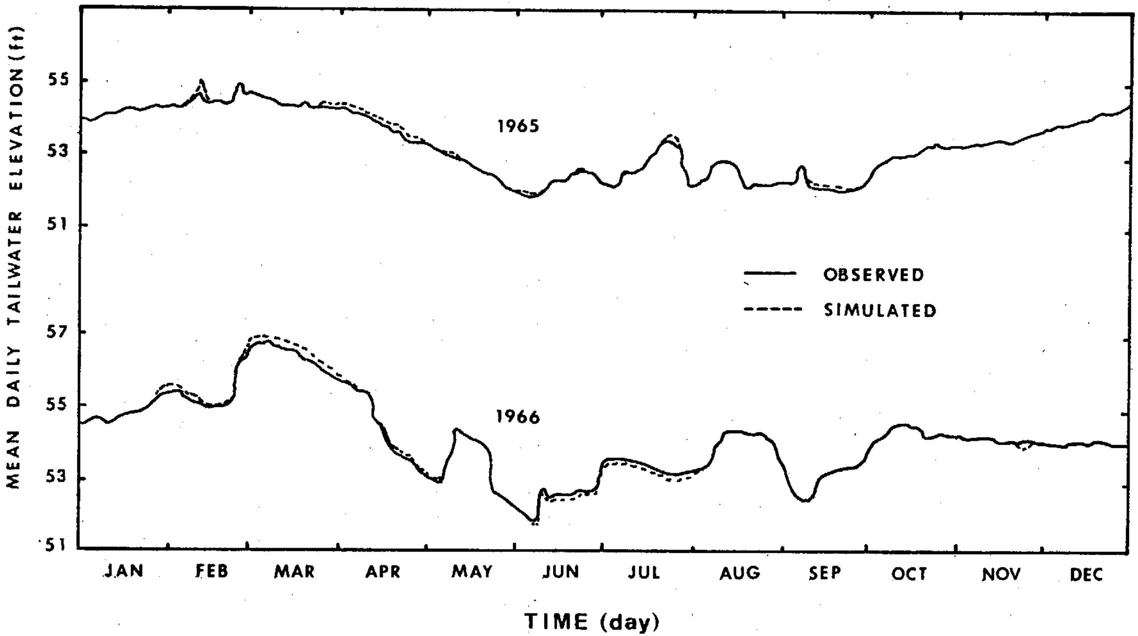


FIGURE 11. SIMULATED AND OBSERVED MEAN DAILY TAILWATER ELEVATION AT STRUCTURE 59

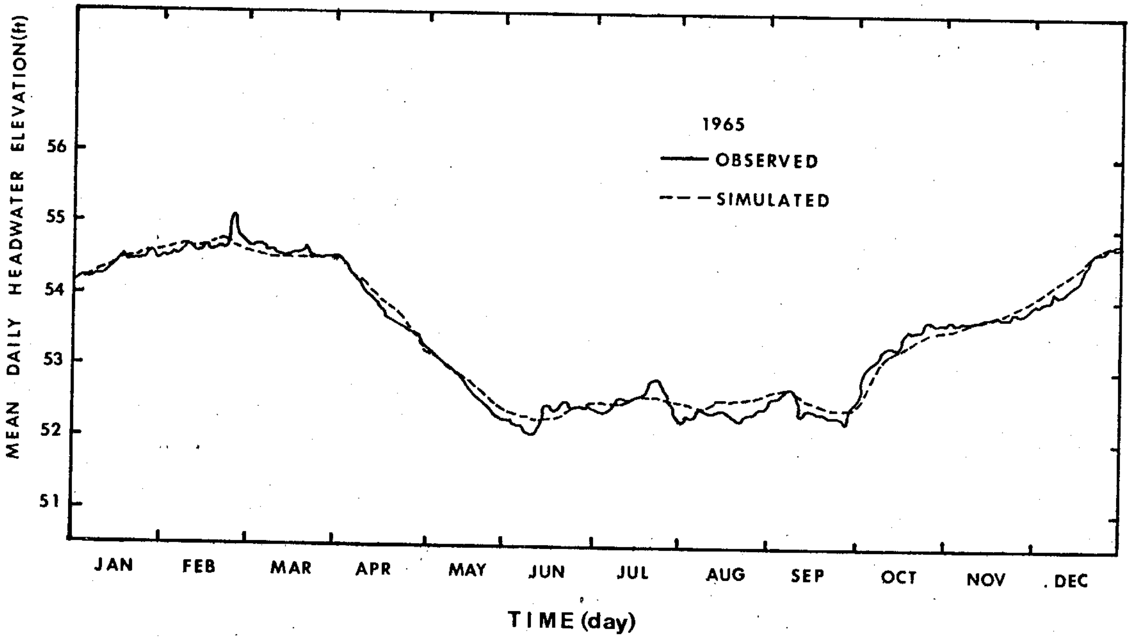


FIGURE 12. SIMULATED AND OBSERVED MEAN DAILY HEADWATER ELEVATION AT STRUCTURE 61

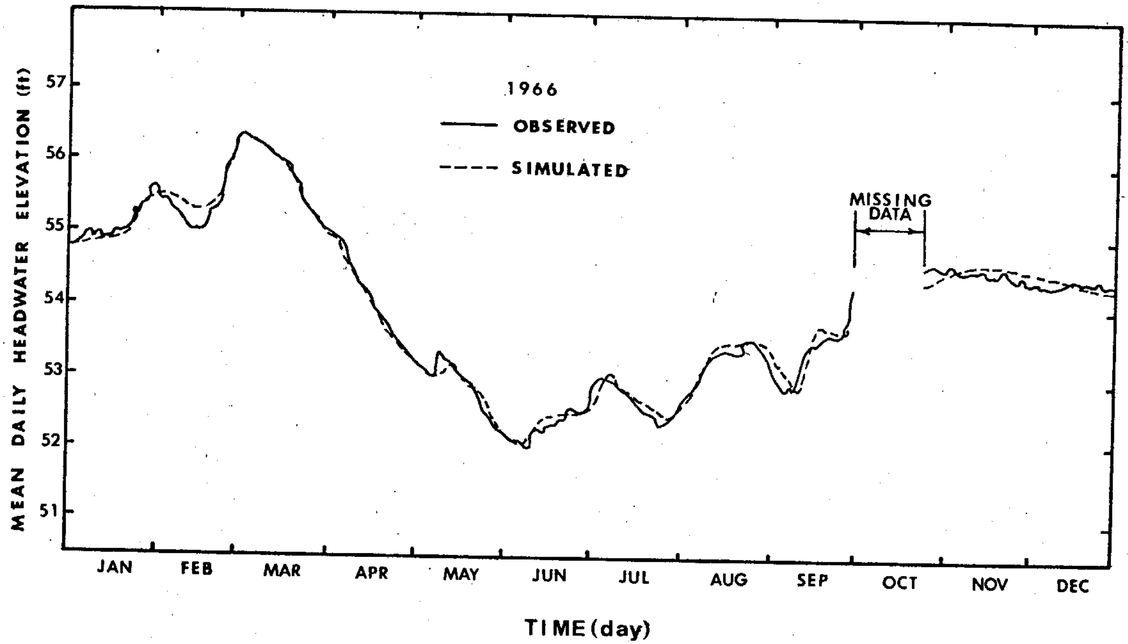


FIGURE 13. SIMULATED AND OBSERVED MEAN DAILY HEADWATER ELEVATION AT STRUCTURE 61